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Interchanging Two Bridge Spans, Wabash R.R. Bridge at Hannibal, Mo.

By F. W. Curtis*

The Missouri Valley Bridge & Iron Co. has just completed some alterations of the Wabash bridge across the Mississippi River at Hannibal, Mo. A very interesting portion of this work was the successful changing of location of a 360-ft., 532-ton draw-span, raising it off its bearing on the old pivot pier, and shifting it westward half its length to a new pivot pier which had been put in place. At the same time a fixed span weighing 270 tons, which had been in place to the west of the draw, was taken out and put in its new location at the east end. Both operations were accomplished by floating the spans from old to new position by means of barges.

The bridge is a combination railroad and wagon bridge, originally built in 1871, the superstructure being rebuilt in 1887 by the Detroit Bridge & Iron Co. It is owned by the Hannibal Bridge Co. and leased by the Wabash R.R. It is of double-intersection Whipple-truss construction and is 1580 ft. long. Calling low-water level of the river zero, the floor of the bridge is at El. 35.7 ft., and high water is 22.5 ft. The greatest depth of the river is 25 ft.

Fig. 1. Moving the 360-Ft. Draw-Span of the Wabash R.R. Mississippi River Bridge at Hannibal, Mo.

(Small picture shows span before moving, but with barges, blocking, etc., in place. Note the old and new pivot-piers.)

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Starting at the Missouri or west shore, the bridge consisted of one span 250 ft., a draw 360 ft., another fixed span 250 ft., and four 180-ft. spans. All the piers were of limestone, resting on 8 courses of 12x12-in. timber grillage, this in turn being supported by foundation piles. These piers are in an excellent condition, not having settled at all; the stones on the outer surface had been bonded together with long driftpins, and the cement mortar was so hard that picks were used to break it up.

Rivermen Complain of Old Bridge

The alterations and changes in the bridge were caused by the complaint of captains of boats and others interested in river navigation, who claimed that the bridge had become an unreasonable obstruction to free navigation by reason of the location of the draw openings, the entire absence of guide fences or sheer booms, and the presence of artificial deposits of stone about the piers of the bridge, which they believed had increased the current through the draw openings to a dangerous extent. Several boats, barges and rafts of logs had in times past struck the piers and either been sunk or badly damaged, though testimony showed the fault in most cases to have been that of the pilot or men in charge of the rafts. A serious accident which brought the matter to a head occurred in June, 1903, when the Mississippi reached the highest stage ever known, 22.5 ft. above low water or only 13.2 below the decking of the bridge. At this time the captain of the "Flying Eagle" endeavored to take a Methodist sunday-school picnic up the river for a boat ride and incidentally show them the Mississippi River at its highest stage. The children were on a barge being pushed by the steamboat. While passing through the draw, the pilot attempted to change the course of his boat, with the result that he lost control and both barge and steamboat were thrown sidewise against the pivot pier and open draw span, both of them sinking. Luckily a large force of Wabash employees were on the draw at the time and aided the passengers on the boat in climbing onto the bridge, so that only four lives were lost.

Formal complaint as to conditions was first filed with the War Department by the rivermen, on Jan. 2, 1905. The Engineer Corps, to whom the complaint was referred for investigation, reported the bridge to be an unreasonable obstruction to navigation and recommended that the draw-span be shifted half its length to the shore: that the 250-ft. span at the west end be cut down to 180 ft. in length and put at the east end of the draw-span to fill up the gap left on account of moving the draw and a new 70-ft. girder-span be placed to connect the draw with the shore; that a new rest pier be built for the east end of the girder and west end of the draw, the old west rest pier torn out and a new pivot pier substituted, the old pivot pier taken down and a new rest pier for the east end of the draw put in, these piers to be in the exact location of the old ones; and that a guide fence be built up the river from the west rest pier, and protection work both upstream and downstream from the pivot pier. A hearing for all concerned was given by the Government Engineers at Rock Island, Ill., on June 6, 1905, and a second at the same place, by request of the Wabash R.R., on Mar. 10, 1906. At the latter, the Wabash R.R. and the bridge company were given until Mar. 15, 1907, to make the changes before mentioned. An appeal was made to the War Department, Wm. H. Taft at that time being Secretary of War, but on July 20, 19[illegible] Rock Island decision was upheld.

Neither the Wabash R.R. nor the bridge [illegible]y having made the changes in the time given th[illegible]uit was filed by the Government on June 4, 1907, t[illegible]pel the changes being made. An interesting feature [illegible]is suit was that the Government was represented b[illegible]ry W. Blodgett as its chief counsel, while his fa[illegible] Wells H. Blodgett, represented the railroad company; the son came out the victor. The case was then appe[illegible] to the Supreme Court, and a decision was rendered May 15, 1911, upholding the previous decision.

Contract for Reconstruction; Work Begins

A contract was let by the Wabash R.R., through A. O. Cunningham, Chief Engineer, to the Missouri Valley Bridge & Iron Co.* (E. H. Connor, Chief Engineer; C. L. Greever, Superintendent of Construction) for making the changes ordered by the government. Plans were drawn by the bridge company, and after having been approved by the Wabash, work was started in the spring of 1912, with T. W. Cartledge in charge of the superstructure work, M. C. Taylor looking after the substructure, and the writer serving as engineer for the Wabash.

Falsework was placed under all panel-points of the 250-ft. west span, and under the west half of the draw-span, and ten 24-in. I-beams were placed over the west rest-pier, supported on the falsework, to carry the ends of both spans. The blocking under the draw-span was so arranged that it could be removed at any time so as to allow the opening of the draw. This kept one channel of the river open for navigation as well as keeping railroad traffic going; no attempt was made to keep the bridge open for vehicles and foot passengers, and in fact the wooden decking was torn out soon after work was started.

Dredging of riprap from around the old rest-pier was started at once, also construction of the new rest-pier near the bank.

West Rest-Pier and Span

Constructing the new west rest-pier was simple: The bottom was dredged out to a solid rock foundation where the pier was to go—only 7 ft. below the water level at the time. A box form for the footing course was well calked and weighted down with rock till it sank in place, and the form then sealed by lowering concrete into it by means of a derrick and a bottom-drop bucket, the bottom of bucket not being opened until on the bottom of river. After this concrete had set the water was pumped from the form and the pier completed. The pier footing is 12x38 ft. and 6 ft. high, the bottom of main body 9 ft. 4 in.x26 ft. with nose on both ends protected against ice with angle-irons, and the top, 7x26 ft. The pier is 29½ ft. high, and contains 411 cu.yd. of concrete. The concrete was a 1:2½:5 mixture, made with sand and gravel dredged from the river and screened to take out some of the sand so as to leave the proper mixture.

While this work was being done, the superstructure of the west span was being taken down, leaving the floor system only in place. The batter posts, some of the bottom chords, and such other parts as were to be redrilled, shortened and changed, were sent to the Leavenworth shop. In the meantime, three 20-ft. panels were cut out

*Leavenworth, Kan.

of the floor system and 5 ft. cut off the stringers at each end of the span, cutting its length from 250 ft. to 180 ft. The floor system was then jacked up so as to allow of placing steel rails under it, parallel with the span, these rails were oiled, and the floor system was pulled together by means of steamboat ratchets. The 70-ft. girders were then swung into place from the shore abutment to the new pier, and the old west span (now 180 ft. long) was rerected.

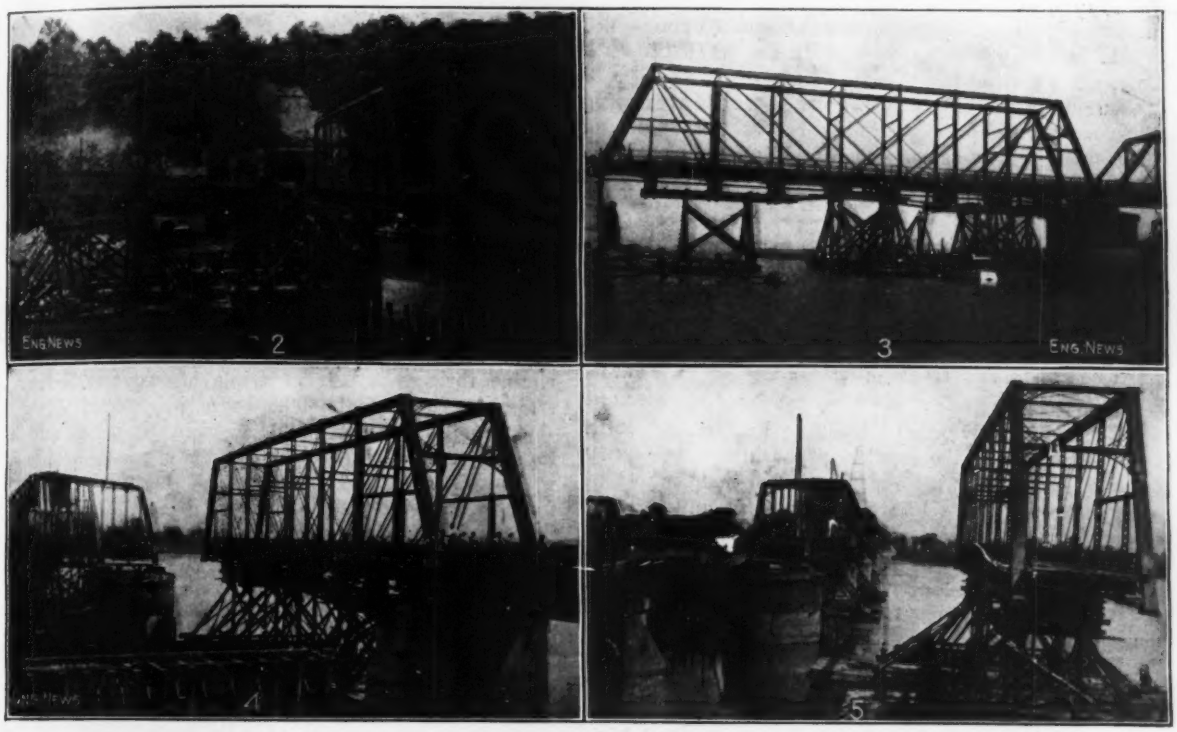
Before this was completed, however, the tearing down of the old rest-pier was commenced. Though the pier was a well built structure, no trouble was encountered in getting it out down to the water level. Various plans were considered for tearing out that portion below water, but it was finally decided to drill and shoot it, then dredge the fragments. Hand drilling was done at first but was too slow, so two steam drills were put to work. A great deal of trouble was experienced in getting the holes down, as the drills tended to strike the slanting seams in the masonry and stick. In shooting the pier, only a few holes were shot at a time, and it was necessary to use great care in shooting, trying to use enough dynamite to break up the pier yet not shoot out the surrounding falsework, which was only 15 ft. away from each side of the pier. No serious trouble to the falsework resulted. The loose rock was dredged out, and, a diver being employed to fasten chains around the separate sticks of grillage and foundation piles, these were pulled out by a derrick.

The old pier contained 439 cu.yd. masonry. The 293 yd. above water was removed in 13 days with an average force of 11 men. For the bottom part it took 9 days to drill and 7 days to dredge the 146 cu.yd. of masonry, and the diver worked 17 days.

Pivot Pier Caisson Foundation

The caisson for the new pivot pier was immediately built and lowered into place, being weighted down with concrete. Sandhogs started work on Dec. 7, 1912, and sank the caisson through 17 ft. of sand, gravel and boulders, reaching shale foundation on Dec. 27. Three shifts a day were used, averaging 11 men to the shift, taking out 654 cu.yd. of material. Bottom was struck at 27.75 ft. below datum, or 35 ft. below water surface (stage of water about 7 ft.). Only light pressure, 17 lb. per sq.in., was used in this caisson. D. W. Hendrick was in charge of the sinking.

Concreting was kept up as the caisson was being sunk, in spite of, or possibly, to be more accurate, on account of the very cold weather prevailing at the time: the river was practically frozen over, with only a narrow channel open and this filled with heavy floating ice; there was ground for fearing that the river might gorge above the bridge and then break loose, carrying the falsework away before the spans were landed on the pier. Concreting was rushed as much as possible, using hot water and salt in the mixing. Owing to local conditions and lack of time it was impracticable to heat the sand and gravel; the thickness of the forms, the great bulk of the pier, and salamanders inside the form and above the concrete, had to be relied upon to keep it from freezing. The forms were left on until spring, and even though the concrete was put in at a low temperature there was no sign of its having been frosted; no scaling whatever appeared, and



FIGS. 2-5. MOVING THE WESTERLY FIXED SPAN, AFTER SHORTENING FROM 250 FT. TO 180 FT.

Fig. 2. Looking toward west bluff and tunnel; barges with towers ready to be put under span. Fig. 3. Looking upstream; barges and blocking in place. Fig. 4. Span just leaving original location; new pivot-pier at left, new rest-pier and new girder span at right. Fig. 5. Span cleared from original location.

holes drilled into it showed it to be as solid as if it had been put in in the summer.

The pier was barely completed when the ice gorged, going out two weeks later and taking most of the falsework. There was ample time for the pier to harden before the load came on it. Work was then stopped for the winter.

The base of the pier is hexagonal in shape, 36 ft. between parallel faces, and 25 ft. high; the upper part is cylindrical, 32 ft. 6 in. in diameter and 29 ft. high, containing 1722 cu.yd. of concrete.

Moving the Spans

On resumption of work in the spring, the sheer boom and protection work above the pivot pier were built, and then preparations were made for moving the spans.

First the tension bars, posts and braces in the center of the draw-span were packed with wooden blocking, bolted into place, so as to withstand the compression which would be caused by supporting the span at the ends only, while moving it.

Four barges, each 70x24 ft., were selected for carrying the draw-span. Framed timber towers were built on them to support the span. The total draft of each barge when loaded was figured as follows:

Displacement per inch $= \frac{61 \times 24 \times 62\frac{1}{2}}{12} =$	7600 lb.	
Weight of barge =	116,400 lb.	
Draft unloaded = 116,400 ÷ 7600 =		15.4 in.
Assumed load from span (1,064,000 ÷ 4) =	266,000 lb.	
Weight of tower on barge =	42,500 lb.	
Load per barge =	308,500 lb.	
Displacement per inch $= \frac{66 \times 24 \times 62\frac{1}{2}}{12} =$	8200 lb.	
Draft due to load = 308,500 ÷ 8200 =		37.7 in.
Total draft =		53.1 in.

It was originally intended to use but two 64x22-ft. barges for carrying the smaller span, which weighed 270 tons. The draft, figured out in a similar manner to that of the draw-span, would have been 57¾ in., and this was so great that a third barge was finally used, in order to make the draft uniform with that of the draw.

On Sept. 23, the barges were floated under the spans and water allowed to enter the holds through valves in the bottom, until the decks were only about 6 in. out of the water. Blocking was then placed between the towers and the floorbeams of the spans, so as to be ready to float the spans off the first thing in the morning. To lift the spans (by pumping the water out of the barges) high enough to clear the old piers required only a few inches on the short span, but 14 in. on the draw, in order that the pinion for turning the span might clear the rack. During the night the river went down 3 in., leaving the blocking loose. As it was urgently desirable to move the spans in as short a time as possible, so as not to delay traffic unnecessarily, an attempt was made to float the spans without tightening up the blocking; but the attempt was a failure, and finally water was let back into the barges and the towers reblocked.

The next morning, Sept. 25, 1913, the water was pumped out of the barges, successfully lifting the spans from off the piers. Three barges with hoisting-engines were anchored above the bridge, and cables from them were run to the barges carrying the spans so as to pull them into their new positions. Front and tail lines were used on the barges to steady them, the idea being that they would float a short distance downstream, then be swung over below their new locations, and then pulled upstream to where they belonged. The wind, however, was upstream, and it was necessary to use a steamboat to pull them downstream and hold them there.

The small span was floated out first and taken a couple of hundred feet downstream, where it was anchored to some piling. Then the draw-span was moved out and put into its new location. This took the balance of the day, so that the short span was not in place and the bridge open for traffic until noon of the next day, at which time the bridge was in as good shape as ever, the draw working perfectly on its new bearing.

During the time the bridge was out of commission, all trains were stopped on the east side of the river, passengers and baggage being transferred by ferry.

J. J. Kelly was in charge of the shifting of the spans. The operation was watched by many officials of the Wabash R.R. and of the Missouri Valley Bridge & Iron Co., besides as many people from Hannibal and the surrounding country as could get away.

Old Pivot Pier and New East Rest-Pier

Work was at once started on removing the old pivot pier, I. F. Lindsay now being in charge of all work.

Falsework was erected as for the other pier, and the pier torn down in a similar manner. It contained 1300 cu.yd. masonry, 8 courses of grillage 45 ft. long by 45 ft. wide, and 250 foundation piles, 10 to 15 ft. long.

A chain was fastened around these piles by a diver and a water jet employed while pulling them, this to loosen them up. An attempt was first made to pull these piles *without* the aid of a jet. A heavy A-frame was erected on the front of a barge and a steel cable passed from the engine through a pair of triple blocks, this giving a straight vertical pull. The barge was pulled down so that the deck was flush with the water and the pile pulled in two, just below where the chain was passed around it, without even starting the balance of the pile. On examination, the pile, although water soaked, seemed to be perfectly sound. The friction around the pile to have been greater than its tensile strength.

The caisson for the new east rest-pier was landed on the bottom of the river 19 ft. below low water, and sunk 39 ft. through the same material as found in the pivot pier, to a level shale foundation. The excavation was 1175 cu.yd. of material; the foundation is 93.27 ft. below base of rail. The shape of the pier is similar to that of the first pier built, the foundation being a rectangle 18 ft. 4 in. by 44 ft. 4 in., 20 ft. high. The main pier is 14 ft. 1 in. by 26 ft. on the bottom, the sides battering to 7 ft. 11 in. by 26 ft. on top. The ends are pointed, and the upstream end is protected by steel nosing. The pier contains 1292 cu.yd. of concrete.

Tearing out the old pier was begun on Sept. 26, and the new pier was completed on Dec. 20.

With the exception of the time used in shifting the spans and a few hours spent one morning in strengthening falsework, there was no interruption of either railroad traffic or navigation at any time.

Now that the work has been completed, it is an open question—and one which time only will solve—as to whether or not the changes made will in any way change the conditions at this bridge as claimed by those interested in river navigation. It is the opinion of the writer that, outside the building of the sheer boom and pier protection, the work was a waste of money, forced upon the Wabash Railroad.

Reports at the National Conference on Concrete Road Building

SYNOPSIS.—Abstracts of some of the more important Committee reports presented at the National Conference on Concrete Road Building held at Chicago, Ill., Feb. 12-14, 1914. Concluded from our issue of Feb. 19.

Methods and Cost of Repairing and Maintaining Concrete Roads

[Each of the three members of the Committee on this subject presented individual papers abstracted below.—Ed.]

In Wayne County, Mich., we have followed two methods in maintaining the concrete roads, the first of which consists in filling the open expansion joint on the roads first built before the development and use of the armor joint, and the second the filling with a mastic such longitudinal and transverse cracks as have developed.

A crew consisting of seven men and a team, provided with a tar kettle, is utilized for the work. The foreman is paid $5 a day, the team and driver $5 a day, the "tar man" $3 a day, two laborers at $2.50 each and two laborers at $2.25 each. The tools used consist of two wire-bristle brooms, a wheelbarrow, a couple of shovels and a tar bucket with a round spout. Tarvia X is now used exclusively. A lighter grade was first tried out, but did not give such permanent results as the heavier grade. Two men sweep all cracks clean with the wire brooms, after which the man with the tar kettle fills the cracks with the tar, which is heated to about 225° F. An excess of tar is poured in so that it extends an inch or so beyond the edge of the crack. It is then allowed to stand in the crack for a few minutes to prevent it from "bubbling" out in case the sand is wet. The sand, which should be dry and coarse, is spread with a shovel over the crack and into the tar, and the whole is left for traffic to iron out. The excess tar and sand is worn away rapidly, leaving a smooth, even surface, over which no jolt is apparent in passing either with a horse drawn or motor driven vehicle. This method of repair prevents the edges of the concrete from spalling and chipping, and no water can get through to the subgrade to freeze and heave in cold weather.

The work is preferably done on hot, dry days. It has been suggested to us that the better time to handle this work would be in the late fall when the cracks would be open the widest, due to contraction, but the results we have secured in the summer months have been so satisfactory that we have not tried out the latter plan. The small pit holes which are due simply to some foreign substance getting into the concrete-like clay, wood, or some fragment of inferior rock which might chance to be a part of the aggregate, are treated in the same manner. As to the cost of this method, the engineering staff, Messrs. George A. Dingman and George F. Key, have presented a report of the repair work done on 51 miles of concrete roads in the fiscal year ending Sept. 30, 1913, showing a total expenditure of $1300.50 for tar, labor and sand.

A percentage was added to cover engineers, inspectors and the depreciation of machinery and building and other "overhead" charges, totaling approximately $1450. The greater part of this mileage was treated for the first time. Once a year is often enough to go over the work and touch it up here and there, as we have found by experience that the bulk of the repairs previously made is intact.

We have not had as good success in treating the entire surface of a road where the concrete from any cause has not stood the wear. One-half mile of concrete on Fort Road, built by Porath & Son, about 800 ft. on Grand River Road, built by the Owosso Construction Co., and 1¾ mile on Gratiot Road, out of the 80 odd miles of concrete road in Wayne County, are not up to the standard of the balance of the construction, and they have been surfaced with Dolarway, a special grade of Tarvia and Tarvia X. The surfaces of Fort and Gratiot Roads were both rough, having been built in cold weather, and opened for traffic before the concrete had thoroughly hardened. Fort Road was surfaced by the contractor before the Board would accept the road and make the final payment. It carries a fairly heavy mixed traffic, not as great, however, as either Grand River or Gratiot Roads. A special grade of Tarvia was used, furnished by the Barrett Manufacturing Co. The road was first swept by a street sweeper, which was followed by men with wire brooms. About ⅓ gal. of bitumen to the square yard was used, and the whole covered with coarse sand and rolled with a 10-ton roller. This repair was made early in the summer and was not touched in any manner for a period of two years. By this time it had scaled off in spots quite badly, and was gone over again in the summer of 1913, using Tarvia X and a washed roofing gravel about ¼ in. in size. We have no record of what the first cost was to the contractor, as we paid nothing for it.

Gratiot Road was surfaced at cost to us by the Dolarway people in the spring of 1912. By the summer of 1913 about half of the original coating had scaled off, at which time we patched it up, using a special grade of Tarvia and Tarvia X. The Tarvia X has given the better result to date. Washed roofing gravel was used with the bitumen. This work was also rolled with a 10-ton roller. Gratiot Road carries an average traffic of over 1000 vehicles a day. (On the last count made one week in August and one week in September the average traffic per day was 1210, of which 36% was motor traffic and 64% horse-drawn vehicles.) The surface of the road is wavy, and we have found it necessary to go over the work two or three times during the summer, touching up small spots here and there where it had scaled off. The cost of the work done by our Board is in the neighborhood of 9c. per square yard.

EDWARD N. HINES,
Chairman Wayne Co. Road Board,
Detroit, Mich.

Repair of Concrete Pavements at Fond du Lac, Wis.

The repairs on our pavements to the present time have been taken care of entirely by contractors who built the pavements. The pavements are guaranteed for five years, and our oldest work was five years old this past summer.

The method of repairing cracks is similar to that in use in Wayne County. Both asphalt and Tarvia have been used, but we find Tarvia gives the better results.

We have had some work of replacing pavement cut out for installing service pipes, such as sewers, gas, and water, also a long piece for installation of telephone conduit.

The excavation is made by the public utility after receiving permit from the Board of Public Works. Excavation is required to be back filled with gravel, sand, or crushed stone to subgrade of pavement. Our men then proceed to trim the edges of hole so walls of cut are as near perpendicular as possible. Mixture same as original pavement is then put in and finished same as original work. Traffic is kept off until concrete is hard. Where the work has been carefully done we have had good success in making such patches. The cost will not exceed that of replacing brick that are cement grouted and, in fact, what records we have would indicate that concrete pavement patches can be replaced cheaper than the brick.

J. S. McCULLOUGH,
City Engineer of Fond du Lac.

Experience at Mason City, Iowa

Mr. Hines' experience as given above I have carefully read; he describes methods I have used on the maintenance of cement city streets, and I consider them the most practicable and economical methods for maintaining a concrete road or pavement. If the roadway or pavement is given a careful inspection at least twice a year and all the defects filled with Tarvia X and coarse, clean sand, free from loam or clay, put on the road or street when the same is perfectly dry, clean and free from any foreign substance, when the temperature is at least 80 to 90° F., good results will follow and the concrete road or pavement will last indefinitely and the cost of maintenance will be reduced to a minimum.

F. P. WILSON,
City Engineer, Mason City, Iowa.

Shoulders for Concrete Roads

With the almost universal tendency in concrete-road construction to keep the width of the concrete as low as practicable, in order to reduce the first cost (per mile) of the improvement of a road, has come, in many cases at least, a neglect of proper consideration of the matter of shoulders. In many cases the neglect referred to has unquestionably resulted in extravagance and waste in the long run, even if it has secured economy in first cost.

While authenticated records covering reasonable periods and concerning the relative expense for the proper maintenance of road crusts and of shoulders, respectively, are not readily available, certain instances of such records, together with the individual experiences related by many authorities, tend to show that the shoulder maintenance costs may be no inconsiderable part of the total maintenance costs of a modern road, and that it may often be real economy to build the road-crusts wider in order to reduce this annual expense for maintaining the shoulders and thus the total maintenance cost per square yard.

The actual width of the concrete roadway to be built must be determined by the traffic conditions to be expected on the road, and if these are sufficiently heavy to warrant so doing, it will probably be economical often to make the concrete itself wide enough to permit the passing of two lines of traffic wholly on its surface and without any necessity for either of passing vehicles to travel off the concrete onto the shoulder. On the other hand, traffic and other conditions may be such as to permit the greatest economy to be had by making the concrete only wide enough for one line of traffic and allowing the occasionally passing vehicle to travel on the shoulder for the moment.

The immediate vicinity of the line of contact between the edge of the concrete and the shoulder material will be found to be the tender region of both the road-crust and of the shoulder. The high rigidity of the concrete road and consequently the greater contrast between such a road-crust and other material usually available for shoulders increase the importance of the careful consideration in the matter.

Where the concrete road-crust is so narrow that vehicles are almost constantly, in avoiding others, passing from the concrete to the shoulder, serious wear of both the road-crust and the shoulder will be found to take place along the region of contact referred to, and this wear is frequently accompanied by the formation of a rut in the shoulder, which holds water in wet weather and thus endangers the foundation of the road. Some of the passing traffic will undoubtedly turn out onto the shoulder further than the rest, but at the extreme outside of the shoulder the travel over it may be so light and infrequent that the natural material there is not strained beyond what might be called "its elastic limit," and there then is no reason, on this account at least, for its replacement at any extra cost.

It will, therefore, be seen that the selection of the material and methods for the construction of shoulders to a concrete road requires the use of something between the natural soil, at the outside extremes, and the concrete itself, that the choices are determined by local conditions, such as availability of different materials and methods, width of the concrete road-crust, amount of traffic, etc.; and that the wearing abilities or "elastic limits" of the shoulder material at any point should be outside or above the stresses to be expected at that point.

The passage of traffic from the pavement to soft adjacent material can, of course, be absolutely prevented, and the protection of the edges of the pavement be had by the installation of a raised edging or curb, as in the cases of most streets. The necessary reinforcement of the shoulder where no raised curb is to prevent traffic on it may be had in various ways. One would, of course, be to widen the concrete road-crust to such an extent as to preclude any possibility of traffic going off it. In doing this the center thickness of the crust need not necessarily be preserved to its edges, but the thickness may be tapered down somewhat at the outsides. Or this widening (and tapering down in thickness) of the road-crust may be done over a portion of what might otherwise be the shoulder and then the reinforcement continued by the use of macadam or pit-gravel or other materials as may be necessary. Again, the width of the concrete road-crust being fixed, the shoulder reinforcement may be had by the construction of pitch concrete, pitch macadam, water-bound macadam (with or without a pitch carpet), etc., adjoining the concrete road, all as may be necessary and desirable under the local conditions. The actual determination must be left to competent parties who will give due consideration to the local conditions and the principles referred to above.

W. W. CROSBY,
E. A. KINGSLEY,
J. H. MULLEN,
Committee.

Mixing and Placing Materials for Concrete Roads

In the construction of concrete roads, probably no really important part of the work is given less consideration, in specifications and in the field, than the actual mixing and placing of the concrete.

The following notes, based upon observations in the field, and the results of tests, were prepared with a view of pointing out the difference between good and bad practice, and to serve as a basis for recommendations covering this phase of the work.

MIXER—The concrete mixer should be of the batch type provided with an automatic water tank, traction drive and power loader. Mixers having a boom and bottom dump bucket of sufficient size to convey one complete batch for placing the mixed concrete, are preferred. However, buckets of other types and sizes, and open troughs can be used and will give good results.

Experience shows that where a tube or spout is used for delivering concrete, it is necessary to mix the concrete too wet for the best results. Another objection to this system is that it may be operated like a continuous mixer, the supposition being that, as the spout is provided with blades and revolves, the materials are mixed in the spout. This spout serves only as a conveyor and cannot to any degree take the place of or serve as a mixer.

The mixer should be provided with a suitable automatic water tank which can be quickly filled and emptied, so that when once determined, the required amount of water can be added to each batch of concrete. A number of so called automatic tanks on mixers are not satisfactory, owing to their limited capacity, delay in filling and emptying, and to the fact that they depend, for successful operation, upon a constant water pressure. Water is pumped on most road jobs, with small gasoline or steam pumps, and it is not possible to maintain uniform pressure in the feed line.

An automatic measuring tank of the required capacity, that may be emptied quickly, connected by means of a comparatively large pipe, with an auxiliary or storage tank of about 50 gal. capacity, would be more satisfactory than the one-tank system commonly used.

Where necessary to keep from cutting up the subgrade and to facilitate moving, the wheels of the mixer should be run on suitable planking. Material of good quality, 3 in. thick by 10 in. wide, in comparatively short lengths with square ends, will be found satisfactory for this purpose.

The power loader or skip should be of sufficient size to hold all the materials required for the batch. In charging the skip a part or all of the coarse aggregate should be placed first, and the cement, fine aggregate, and remaining coarse aggregate, if any, on top of this. All the cement should be in the skip before the last of the aggregate is added. If charged in this manner there will be less tendency for the materials to stick in the skip when emptying and less loss of cement on windy days. A systematic method of loading the skip will also serve as a check on the right amount of material being placed each time.

The filling of the skip is accomplished in practice in two ways—by the use of wheelbarrows and by shoveling direct from the supply piles into the skip. This latter practice, however, should be discouraged, for accuracy is impossible, and it encourages carelessness. Besides, the entire loading gang loses time waiting while the skip is raised and lowered.

MIXING CONCRETE—No important operation is given less attention than the mixing of the concrete. Frequently in practice, the time the materials remain in the drum is governed only by the speed of the gang charging the skip. At times the batch mixer is operated practically as a continuous mixer, by fastening the discharge scoop or spout so that there is a constant flow of concrete from the mixer.

The quality of concrete is largely dependent upon thorough mixing. To insure thorough mixing the revolutions per minute of the mixer and the time the complete batch, including water, remains in the mixer, should be specified. To specify either the r.p.m. or the time is not sufficient as it is plainly shown by field observations that there is not a direct relation between the two. Upon inquiry of the manufacturers it was found that the r.p.m. of the drum of batch mixers varies for different makes and for different sizes of the same make.

Because of the shape and arrangement of the interior of the drum, the rate at which a given mixer is operated has a direct bearing upon the quality of the concrete; therefore the drum should be operated at approximately the speed at which the manufacturer claims the best results would be obtained.

It is recommended that all specifications contain a clause to the effect that all the materials in any one batch—including the water—should remain in the drum of the mixer at least 45 sec. before any of the concrete is discharged.

In all cases the drum should be completely emptied before the next skip of materials is dumped into the mixer. This is a source of constant controversy between the engineer or inspector and the contractor, especially where the mixer is used

equipped with an open trough, or with a boom and bucket when the bucket is not large enough to hold the whole mixed batch.

The operator should start the water into the drum as soon as the skip is in position to dump—that is, it is not necessary to mix materials dry before adding water.

CONSISTENCY—The practice on road work is to mix concrete entirely too wet. This causes a separation of the coarse materials from the mortar, resulting in stony pockets throughout the concrete. Where the concrete is mixed too wet, it is practically impossible to obtain and hold the required crown, and stony patches frequently appear on the surface after it has been finished, owing to the flow of water and mortar to the sides. In striking off and floating concrete mixed with an excess of water, it is also practically impossible to obtain a surface of the desired character, as the excess of water collects in and hides depressions and other inequalities in the surface, which cannot be corrected as they are not apparent until after the water has evaporated.

In addition to the difficulty encountered in getting wet concrete into the pavement in a manner which will give the best results, laboratory tests and results observed in the field show that, other things being equal, a wet mixture is of inferior strength and quality to concrete of a medium wet consistency. The marked effect in the strength of concrete that a variation in the consistency produces, is shown in the table of results of tests made in the Sheffield Scientific School of Yale University by Charles J. Robison, in "Engineering News," May 22, 1913, p. 1063.

EXPANSION AND CONTRACTION—As yet there is not sufficient data on which to base a definite conclusion relative to the influence of the consistency of concrete upon the expansion and contraction; however, information at hand seems to show that this action is influenced very largely by the consistency.

Owing to the methods employed in striking off and finishing the surface of concrete pavements, there is little likelihood of mixing the concrete too dry. Concrete mixed with an excess of water is easier to mix, to handle, to place and to finish, than concrete of the proper consistency, which explains the tendencies toward wet mixtures.

The amount of water in the concrete should be such as to cause it to settle to a flattened mass when dropped from the bucket, but not sufficient to cause it to flow readily on the subgrade. The consistency should be such as not to require tamping, but not so wet as to cause a separation of the mortar from the coarse aggregate in handling and placing. If there is an excess of water on the surface, or the mass has a tendency to flow or settle out of position after being floated, the concrete has been mixed too wet.

PLACING CONCRETE—Just before placing concrete, the subgrade should be well sprinkled so that it will not absorb moisture from the concrete, and should be checked by the engineer or inspector to make sure that the required thickness of concrete can be placed. Checking the subgrade may be done by the use of a straight-edge or line, resting on, or stretched from the top of the side forms.

Probably the most satisfactory method of depositing the mixed concrete in position on the subgrade is by means of a bottom-dump bucket running on a swinging boom free from the mixer. The boom can be swung over any position on the subgrade, and the bucket can be run out and dumped at any point along the boom. Such equipment does not depend for economic operation upon consistency and the concrete can be easily handled and deposited at any place on the subgrade necessitating but little handling with shovels. Between each change in position of the mixer a section, the full width of the pavement (up to 1½ times the length of the boom, and as long as the boom), may be concreted.

When concrete is placed by means of the open trough, care must be exercised to see that it is not mixed too wet, as this method of distribution depends for economic operation very much upon the consistency of the concrete. An open trough is necessarily much shorter than a boom and is more limited in action, which requires that the mixer be moved ahead more often, also that more of the concrete after being deposited on the subgrade be handled with shovels.

When a batch mixer, not having a boom and bucket or open trough, is used, the concrete should be handled in wheelbarrows or hand carts wheeled on suitable runways. Placing concrete by means of horse carts should not be permitted as they cut up the subgrade badly and to get the concrete into final position requires a maximum amount of shoveling.

Whatever the method of conveying and placing, the concrete should be deposited upon the subgrade to the required depth and for the entire width of pavement in as nearly one operation as practical so as to reduce handling to a minimum. If it is necessary to handle a large amount of material with the strike-board it will ride the concrete and thus produce a wavy, uneven surface. Attention to this matter will materially lessen the difficulty and labor in striking off the surface and will result in a very much better finish.

The excess of coarse material that accumulates in front of the strike-board should be uniformly distributed over the surface of the pavement and not left in narrow strips across the section, or placed along transverse expansion joints. An ordinary garden rake will be found useful in distributing the material that accumulates in front of the strike-board. When the strike-board is within 2 ft. of an expansion joint the excess material that has accumulated in front of it should be removed with a shovel and deposited on the subgrade in the next section. Special care should be taken to place good, dense concrete along expansion joints, and all inequalities at the joints, including any small holes left after the removal of the installing device, should be filled with a mortar composed of one part cement and not more than two parts fine aggregate.

The man handling the strike-board should follow closely behind those placing the concrete, for keeping up with the strike-board will materially assist those placing concrete in depositing the required amount of material.

The placing of concrete should be a continuous operation, and stops should be made only at expansion joints. In case the mixer breaks down concrete should be mixed by hand to complete the section, or an expansion joint should be placed at the point of stopping work. Any concrete in excess of that needed to complete the section, when work is discontinued, should be spread out in a thin layer, not exceeding 2 in. in thickness, over the subgrade in the next section and not piled up along the expansion joint.

The strike-board should be cut to conform to the crown of the finished surface of the pavement and should be of sufficient strength and stiffness to show no deflection at the center when supported at the ends, nor a material bowing out of alignment when in use. It should be about 2 ft. longer than the width of the road, protected on the bottom edge with a metal facing and provided at each end with suitable handles.

For roads up to 12 ft. in width two 2x6-in. planks, dressed on one side and both edges, spiked together, make a good strike-board, and for roads 12 ft. to 20 ft. in width two 3x8-in. or a 2x10-in. and a 3x10-in., spiked together, will be found satisfactory. Two planks, well spiked together, make a better strike-board and one less likely to warp out of shape than a single piece. The strike-board should not be so heavy that it cannot be easily handled by two men, and to reduce weight and increase stiffness, it is advisable to work over 20 ft. in width to use a strike-board composed of boards 2 in. or 3 in. thick, stiffened by trussing, rather than obtain the required stiffness by the use of heavier timbers.

In striking off a pavement 16 ft. or less in width, where the concrete has been properly placed, two men should be able to handle the strike-board, but for wider pavements the services of a third man will be required to assist in pulling the board forward by means of a drag line fastened at its center.

The strike-board should always be worked forward about perpendicular to the axis of the roadway and as it is moved ahead should be sawed back and forth across the road.

To produce the desired effect will require that the strike-board be passed over the surface of the concrete two or more times.

Though it will be necessary for the workmen to get into the concrete soon after it is deposited on the subgrade, it is desirable that all unnecessary walking in and wading through the concrete be avoided, and under no circumstances should any workmen step upon the concrete after it is first struck off.

PAUL D. SARGENT,
ARTHUR H. BLANCHARD,
C. W. BOYNTON.
Committee.

Thickness, Crown and Grades for Concrete Roads

THICKNESS—The thickness of the concrete pavement is controlled by many factors, such as condition and character of the subgrade, drainage, traffic, climatic conditions, width of pavement, etc. Three distinct types of cross-sections are in general use:

A. Uniform thickness of concrete for all widths of roadway, and consequently with the same amount of crown in the foundation as in the surface.

B. Roadways in which the concrete is thicker at the center than at the edge but in which some crown is given to the foundation.

C. Concrete roadways in which the concrete is thicker at the middle than at the edge, but which are built upon a flat subgrade. This type seems to be fast gaining in favor.

The thickness of concrete pavements varies from 6 to 8 in. at the crown line and from 5 to 8 in. at the edge. The necessity for the greater thickness is perhaps more urgent in the northern than in the southern parts of the United States, because of the greater extremes of temperatures. Because of the growing tendency to increase the size of the loads, it seems reasonable to suppose that we have not yet reached the extreme limit of thickness required.

Because of the fact that the members of this committee have had their attention directed chiefly to the conditions obtaining in the Northern states, it is not without some misgivings that they suggest the following as conforming to the best practice:

1. Where the width of roadway is not greater than 16 ft. with a porous subsoil, subgrade in good condition and well packed and with loads not exceeding six tons, including the vehicle, a thickness of 5 in. at the edge of the slab and 7 in. at the center is sufficient without reinforcement, using slabs not greater than 16 ft. square.

2. On clay soil and the above general conditions artificial drainage should be provided and reinforcement used (¼ to ½ lb. per sq.ft.), placed 1½ to 2 in. from top surface; and slabs made not larger than above and not less than 6 in. thick at edge of slab and 8 in. at the center.

3. Where the roadway is in excess of 16 ft. with loads of 6 to 10 tons, but with the other general conditions as described in (1), the crown of the pavement should have a thickness of 8 in. and the edge 6 in. Under such circumstances reinforcement will usually be considered necesary and economical.

CROWN—Unlike some types of pavements, those of concrete are undamaged by water, unless it should find its way to the subgrade. However, since even thin sheets of water or ice on the surface of any pavement are objectionable, a sufficient crown must be provided to insure the unwatering of the pavement. Theoretically, with a perfectly surfaced concrete, a very slight side fall only is required to accomplish this. Practical experience in such construction, however, has demonstrated the great difficulty in preventing small imperfections and depressions in the surface; therefore sufficient cross fall must be provided to insure their drainage. The difficulty of accomplisning this is increased because of the suspension of large amounts of dust in the street water.

Attention is directed to the fact that on city streets with side curbs we face a different problem from that found in a narrow country road without curbs. In the city street the crown must not only be sufficient to cause the water to run from the center to the side of the pavement, but also enough to insure that an undue proportion of the pavement shall not be covered by water. This requirement is aided if the pavement is given a "peaked" crown, i.e., say, ⅞ of the total rise at ¼ distance from center to curb, and ⅝ at half the distance.

The small crown that may be properly used on concrete pavements is justly considered an important merit of this type of pavement. Data have been collected which seem to show that in the case of uniformly thick concrete pavements the central longitudinal failures has varied directly with the amount of crown. To a lesser extent this would be expected of the other types, thus furnishing an additional reason for the adoption of a minimum crown.

It appears that the minimum crown used on streets is $^1/_{60}$ of the width of the road and the maximum $^1/_{40}$. With few exceptions, the crown varies between $^1/_{70}$ and $^1/_{100}$ of the width. When the climatic conditions will allow, it is our opinion that a crown of $^1/_{100}$ of the pavement width represents present best practice. The use of the high crowns has in large part resulted from past experience with other types of pavements where high crowns were really required. We are of the opinion that in future construction, as methods of handling materials are improved and contractors become more experienced, engineers should not hesitate to specify as low a crown as $^1/_{100}$ part of the width. Indeed, this has already been done to some extent.

GRADES—The committee have been unable to find any reliable data on the relative slipperiness of concrete pavements. It is greatly to be regretted that such information is not at hand. In view of the ease with which tests of this nature could be made and the importance of such knowledge in the design or selection of pavements, we recommend to this convention that experiments along this line be undertaken.

From such observations as we have been able to make, we see no reason why a concrete pavement, properly finished, may not be used on grades as steep as any other hard-surfaced pavement, and on steeper grades than either wood or asphalt. On steeper grades than 3 or 4% shallow corrugations in the surface will be found of much assistance—in fact, with grades of 5% and over, a real necessity.

LEONARD C. SMITH,
EARLE R. WHITMORE,
T. R. AGG.
Committee.

Cost of Constructing Concrete Roads

The committee has confined itself to averaging the cost of work reported in the different states, confining the comparison to the one course construction. As follows:

AVERAGE COST OF ONE COURSE CONCRETE ROADS

State	Cost per sq.yd.	State	Cost per sq.yd.
Arizona	$1.20	Montana	$1.83
Arkansas	0.90	Nebraska	1.40
California	1.17	New Jersey	1.12
Colorado	1.25	New York	
Connecticut	1.32	North Dakota	1.26
Delaware	1.61	Ohio	1.22
Idaho	1.09	Oklahoma	1.05
Illinois	1.01	Oregon	1.39
Indiana	1.23	Pennsylvania	1.16
Iowa	1.11	Tennessee	1.20
Kansas	1.08	Washington	1.31
Maine	1.48	West Virginia	1.32
Maryland	1.21	Wisconsin	1.06
Massachusetts	1.29		
Michigan	1.27	Average Total	$1.24
Minnesota	1.05		
Missouri	1.17	Weighted Total Av.	$1.19

In general, it is to be noted that the labor cost is about 44% while the materials cost is about 56%; and that the cement cost is a little over one-fifth of the total cost of the road, while the aggregate cost is a little over one-fourth of the total cost.

A. N. JOHNSON,
JOSEPH HYDE PRATT,
ALBERT REICHMANN,
Committee.

The Contraction and Expansion of Concrete Roads

1. THE EFFECT ON EXPANSION AND CONTRACTION OF TEMPERATURE CHANGES IN THE CONCRETE

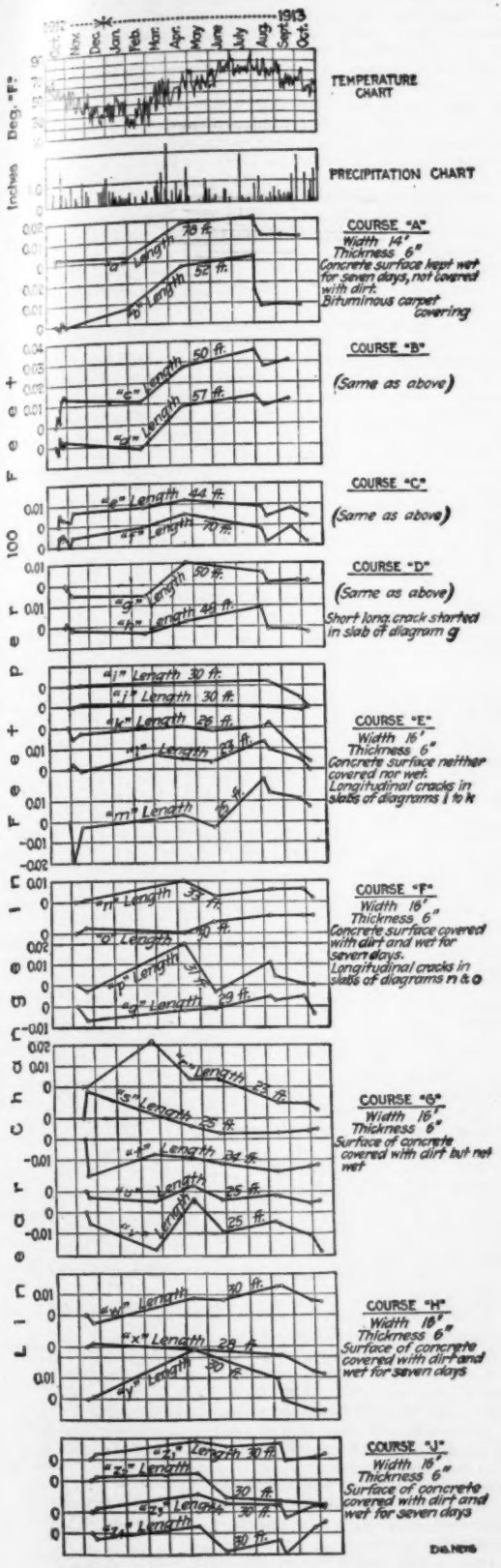
It is generally considered that the variation in the temperature from season to season tending to cause a change in length, combined with unequal frictional resistance between the concrete and the sub-base, is the primary cause of cracking. The change in the length of concrete due to temperature if free to move ranges from 0.00000805 to 0.000004355 per degree F. per unit length, the accepted value being about 0.000006. Assuming a normal range of temperature of 90° F., the movement which would occur without restraint is about 0.00054 per unit of length.

In practical work, however, several elements enter tending to modify the temperature effect. A reduction in the moisture content of concrete would cause a contraction, while an increase would cause expansion, thus aiding or counteracting the effect of temperature. The friction on the subbase in the case of a road will always reduce the movement. In some recent experiments made by Henry S. Spackman upon experimental slabs of 1:2:3 concrete 18 in. wide, 6 ft. 6 in. long and 6 in. thick, on a clay subsoil, a total movement of 0.00017 per unit length was observed under a change in temperature of 65° F. between July and November. The theoretical temperature movement for this range would be 0.000384 per unit of length.

Measurements made by the Bureau of Standards on a concrete road on Morris Turnpike, near New Village, N. J., covering a period from October, 1912, to October, 1913, and in Nazareth, Penn., from June to October, 1913, show that the linear change in the concrete is not in accord with the temperature change. These measurements were made by stretching an invar tape, graduated to meters, along the road spanning several slabs and reading every second meter, the figures recorded as slab changes represent changes between the points nearest the ends of the slab. Readings were recorded to hundredths of a millimeter (0.00039 in.) This is illustrated in the accompanying cut, in which the linear changes occuring in a number of slabs have been plotted.

There was an elongation from November to April, when it reached a maximum, followed by a shortening until August, and this is followed by an elongation until October.

During this period there was a mean temperature range from 16° to 84° F.


1913
Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct.
Deg. "F."
TEMPERATURE CHART
Inches
PRECIPITATION CHART
COURSE "A"
Width 14'
Thickness 6"
Concrete surface kept wet for seven days, not covered with dirt.
Bituminous carpet covering
"a" Length 78 ft.
"b" Length 52 ft.
COURSE "B"
(Same as above)
"c" Length 50 ft.
"d" Length 57 ft.
COURSE "C"
(Same as above)
"e" Length 44 ft.
"f" Length 70 ft.
COURSE "D"
(Same as above)
Short long crack started in slab of diagram g
"g" Length 50 ft.
"h" Length 45 ft.
COURSE "E"
Width 16'
Thickness 6"
Concrete surface neither covered nor wet.
Longitudinal cracks in slabs of diagrams I to K
"j" Length 30 ft.
"i" Length 30 ft.
"k" Length 26 ft.
"l" Length 23 ft.
"m" Length 15 ft.
COURSE "F"
Width 16'
Thickness 6"
Concrete surface covered with dirt and wet for seven days.
Longitudinal cracks in slabs of diagrams n & o
"n" Length 33 ft.
"o" Length 30 ft.
"p" Length 29 ft.
"q" Length 29 ft.
COURSE "G"
Width 16'
Thickness 6"
Surface of concrete covered with dirt but not wet
"r" Length 23 ft.
"s" Length 25 ft.
"t" Length 24 ft.
"u" Length 25 ft.
"v" Length 25 ft.
COURSE "H"
Width 16'
Thickness 6"
Surface of concrete covered with dirt and wet for seven days
"w" Length 30 ft.
"x" Length 28 ft.
"y" Length 30 ft.
COURSE "J"
Width 16'
Thickness 6"
Surface of concrete covered with dirt and wet for seven days
"z1" Length 30 ft.
"z2" Length
"z3" Length 30 ft.
"z4" Length 30 ft.
Feet per 100 Feet
Linear Changes in Concrete Slabs

Considerable rain fell during the latter part of March and the early part of April. The contraction from April to August is contrary to the condition which should result from a rise in temperature, and this can only be explained by a drying out of the concrete sufficient in amount not only to overcome the expansion due to a temperature increase of 20° F., but also to show a substantial contraction. From August to October the slab shows expansion, which again can be explained by an increase in moisture content, as there is no rise in temperature.

2. THE EFFECT ON EXPANSION AND CONTRACTION OF VARIATION IN THE MOISTURE CONTENT OF CONCRETE

It has been definitely established that with an increase in moisture content there is an expansion of the concrete, and with a decrease in the moisture content a contraction. These phenomena are apparently true for all concretes at all ages.

Experiments with sections of a top coat of a cement walk which had been laid 20 years showed that it expanded when immersed 0.0005 per unit of length and contracted the same amount when again dried.

Experiments with a section of stucco two years old, from a brick house, showed a linear expansion when immersed of 0.0008 per unit of length in four days, and a return to its original volume when dried.

Experiments with a section of cement walk in which a sample of both the top course and base were bound together showed that upon immersion the base reached its maximum expansion in 15 minutes while it took the top coat three days to reach the same expansion.

This is interesting partly because of the evidence of alternate bending stress in the concrete due to the more rapid expansion of the lower layer and partly because of the ultimate agreement in expansion of the top and bottom portion. This cement sidewalk was in good condition after 20 years' service.

All these tests indicate that the effect of moisture content is very much greater than the effect of temperature change, and may be sufficient to cause a stress in the concrete opposite to that which would be caused by a normal temperature change.

A variation in the quality of the concrete will cause a variation in the tendency to expand and contract with change in moisture content, as dense mixtures absorb water less rapidly than porous mixtures.

If concrete of two qualities in a road were exposed to an equal amount of moisture for a short period, the more porous of the two would have a tendency to expand the most and would have the less strength to resist the stresses set up. Assuming only a slight change in temperature and that the frictional resistance on the sub-base was uniform, the more porous concrete would have to move the greater distance, higher tensile stresses would be set up upon drying out and it would be the more liable to crack.

If one-course work concrete of two qualities in the same slab in a road were similarly exposed, the same tendency would be exhibited and this may, in certain cases, be sufficient to cause a separation of the two concretes, or may cause a number of small cracks.

If in two-course work the base is of a more porous concrete than the top, the tendency would be to have unequal expansion and contraction in the slab, due to different moisture content in these mixtures causing the bottom to move more than the top, or vice versa. Also if the concrete road is subject to a heavy rain for a considerable length of time the bottom will be exposed to moisture in the ground for a longer time than the surface, and the surface will be more or less dried, due to exposure to the sun and consequently the top would tend to contract while the bottom would still be expanding.

3. THE EFFECT ON EXPANSION AND CONTRACTION OF VARIATION IN THE CONDITION AND THE CHARACTER OF THE SUB-BASE

The condition and character of the sub-base would affect the support of the road; the amount of moisture transmitted to and from the concrete; the action of frost; and the friction on the bottom of the slab. All of these factors affect the expansion and contraction of the concrete.

If the slab of concrete is not uniformly supported, sections may be stressed, causing tension or compression in an amount sufficient to exceed the strength of the concrete, resulting in cracking. A condition illustrating this point may be found where a concrete road is built over an old stone road bed of less width than that of the concrete slab. The foundation at the sides, which is not so compact as that at the center, will settle more, causing expansion in the surface of the concrete

slab possibly sufficient to cause a crack. While there are no experimental data available definitely establishing this point, it is believed the longitudinal cracks in Course "C," New Village, N. J., may be thus acounted for.

The amount of moisture transmitted to and from the concrete through the sub-base depends upon the porosity and density of the material in the sub-base. The action of frost can only occur in a sub-base containing moisture in localities in which the temperature gets sufficiently low to freeze to a depth of the sub-base. The effect of freezing and thawing is to place the slab in stress which may cause cracking.

The friction on the bottom of the slab is dependent upon the condition of the sub-base. The effect of this friction is to restrain movement in the concrete; therefore, the tendency to volumetric change due to change in moisture content and thermal change would be restrained.

The longitudinal grade of the road would affect the friction on the bottom of the slab. On a 5.2% grade on Morris Turnpike a slab 30 ft. long was observed to have moved down hill, the joint at the upper end of the slab being decidedly open while the lower end of the slab slid upward on the adjoining slab. The joint face in this case was not perpendicular to the slab.

Application of Present Knowledge to the Prevention of Cracks in Concrete Roads

A—CRACKS

Is it possible to prevent all cracking of concrete in roads by a proper application of engineering principles?

Your Committee believes that with a proper understanding of the physical phenomena affecting the expansion and contraction of concrete in roadways, cracking can be eliminated.

(1) When do cracks usually occur?

Longitudinal cracks usually do not occur until seasonal changes.

Transverse cracks may occur at any time, but the majority should occur during the first dry season.

(2) What is the cause of transverse cracks?

Transverse cracks are probably due to a favorable combination of moisture content in the concrete and atmospheric temperature conditions, together with a restraint induced by the condition of the subgrade.

They may, however, be caused by an unstable foundation.

(3) What is the cause of longitudinal cracks?

Longitudinal cracks are probably in the majority of cases caused by an unstable condition of the sub-base, although they may be caused by a favorable combination of moisture content in the concrete and atmospheric temperature conditions, together with a restraint induced by the condition of the subgrade. The latter is particularly effective if the sub-base is crowned.

(4) What is the cause of diagonal cracks?

Diagonal cracks are probably most commonly caused by a combination of moisture content in the concrete and atmospheric temperature, together with a restraint induced by the condition of the sub-base. In rare cases they may be caused by the above combination and longitudinal restraint along one side when abutting a rough structure.

B—EFFECT OF QUALITY ON CRACKING

(5) What mixture of concrete should be used?

The best mixture is the one which is the most dense and which will reduce to a minimum the absorption of moisture, but it should also have a strength of at least 1500 lb. per sq.in. at the end of 28 days.

(6) How should the concrete be mixed?

It should be mixed in such a way as to give a product of uniform quality.

(7) How should the concrete be placed?

It should be placed in such a manner as to give a uniform quality of concrete in place and to prevent loss of water.

(8) How should the sub-base be treated before placing the concrete?

The sub-base should in all cases be thoroughly wet so as to prevent absorption of moisture from the concrete and thus cause a rapid drying out of the concrete.

(9) How should the concrete be cured?

The concrete should be cured in such a way that it will retain its own moisture and receive sufficient additional moisture until it is strong enough to resist the shrinkage stresses induced by drying out.

C—RELATION OF SLAB LENGTH TO CRACKING

(10) What is the maximum permissible length of plain concrete slabs?

The maximum permissible length of slabs, of similar design, on a firm foundation, on a light or flat grade will vary with the climatic conditions. In localities where there is little rainfall, as in Arizona, or the San Joaquin Valley in California, the slab length probably should not exceed 25 ft. This length has been determined by an analysis of a report by A. B. Fletcher, State Highway Engineer, of California, on cracking of a monolithic road, together with a knowledge of the results of measurements obtained by the Bureau of Standards.

In localities where the rainfall is intermittent and the ground water rises in cool seasons and lowers in warm seasons, similar to conditions which obtain east of the Mississippi River, the slab may vary in length from 20 ft. to a continuous slab. While the 30-ft. slab may be regarded as a minimum length, from economical considerations, the results of experiments indicate that cracks are quite likely to occur under certain conditions even in this length.

(11) Does the length of the slab affect longitudinal cracking?

The data available to the Committee would indicate that the length of the slab for the same subsoil conditions does not affect longitudinal cracking.

(12) Is it necessary to vary the length of the slab for concretes of different qualities?

The length of slab should vary with the quality of the concrete. In a short time a denser concrete will absorb less moisture and consequently will have less tendency to move; it will also have greater strength to resist stresses induced by restraint of the subbase. A dense concrete is particularly of value where subjected to intermittent wetting and drying. In a dry climate where plenty of water is not available for curing the slab length should not exceed 20 ft.

(13) Should the length of the slab be the same for all subsoil conditions?

If the subsoil is of unstable material the length of the slab must be shorter than for a condition in which the subsoil is stable.

(14) Is it necessary to vary the length of the slab with a variation in the character of the sub-base?

If there is a distinct variation along the length of the road in the materials of which the sub-base is constructed a joint should be placed where the change takes place. This is necessary because of the change in the frictional restraint offered by different materials and also because of the difference in the porosity which changes the moisture conditions.

(15) Is it necessary to vary the length of the slab with a variation in the condition of the subgrade?

The length of the slab should vary with the condition of the subgrade. If the subgrade could be made very smooth, so that a more or less uniform frictional resistance would be obtained throughout the length of the slab, it would perhaps be the most desirable condition, but as this is not obtainable, it is believed that a more or less uniformly rough subgrade is preferable to a so called "smooth" subgrade.

(16) Is the length of the slab dependent upon the grade of the road?

If the grade is steep the slab should be shorter in length than that used on a flat grade. A joint should be placed at all decided changes in grade.

(19) Is it necessary to vary the length of the slab if a bituminous carpet covering is applied?

The experimental results indicate that there was greater movement of the concrete covered with a bituminous carpet than with concrete uncovered. The reason for the greater movement noted is probably to be explained by the fact that a dark surface absorbs more sun heat and the bituminous coating holds for a longer period any moisture which gets beneath it.

(20) May the length of the slab be made greater for a greater thickness of slab?

The length of the slab may be made greater for a greater thickness of slab. The increased friction caused by increased weight is proportionately less than the increased ability of the concrete to resist stress.

(21) Should the length of slab be different for different traffic conditions?

It is not believed that traffic need be considered in determining the length of the slab.

(22) Does the type of joint affect the length of slab?

A tight joint would permit of a longer slab. It is not known, however, that this factor is sufficiently appreciable to be considered in determining the length of the slab.

(23) What is the relative likelihood of longitudinal cracking in slabs of different design but of uniform cross-section?

A concrete road laid on a crowned sub-base would be the most liable to crack longitudinally and on a dished sub-base the least liable to crack. With the dished sub-base the concrete contracts with the effect of gravity and, therefore, is stressed the least.

(25) Does the width of the slab affect transverse cracking?

It is not believed that transverse cracks are in any way affected by the width of the slab.

(26) Is it necessary to vary the design of the cross-section of the slab with the relative position of the longitudinal drain?

The slab should have its greatest thickness at the center no matter where the longitudinal drain is placed.

(27) Is it necessary to vary the design of the cross-section of the slab with subsoil conditions?

Over an unstable subsoil the slab should be made of greater thickness throughout the width than over a stable subsoil, with the greatest thickness at the center.

(29) What is the preferable design of cross-sections for different climatic conditions?

The dished is the preferable design under all climatic conditions, especially in arid regions.

(30) Is it necessary to vary the cross-section of a slab with the season when constructed?

The dished sub-base is the preferable cross-section for all seasons and especially so if the concrete is laid during the dry season.

(31) What should be the relation between the thickness at the center and the width of the concrete road?

The wider the road the thicker should be the slab.

(33) Is it necessary to vary the design of the cross-section with the traffic conditions?

A concrete road which must withstand heavy and concentrated traffic must be made thicker than for light traffic.

E—THE VALUE OF REINFORCEMENT IN PREVENTING CRACKING

(34) When should reinforcement be used?

Reinforcement will be of value:

(a) When the foundations are uncertain;

(b) In regions where there is little rainfall and long or wide slabs are desired;

(c) In regions where there is insufficient water for curing;

(b) When it is necessary to have a thin slab;

(e) When a decided change in grade occurs;

(f) When the movement of the concrete is restrained at street intersections or on curves, if joints cannot be placed properly;

(g) When the width of the slab is greater than 25 ft.;

(h) When it is necessary to crown the subgrade and the slab cannot be made thicker at the crown than at the sides.

(35) What type of reinforcement should be used?

Only fabricated steel should be used and that form which will give the greater distribution.

(36) How much reinforcement should be used?

The quantity of reinforcement can only be determined by an analysis of the conditions affecting each specific case.

(37) What is the proper position of the reinforcement in the slab?

If uncertain subsoil conditions tend to cause a settlement on the sides of the road, which is the most common condition, the reinforcement should be near the top. If settlement is apt to occur within the center of the slab the reinforcement should be placed near the bottom, and where the whole of the sub-base is uncertain the reinforcement should be placed near both top and bottom.

Reinforcement to resist stresses due to temperature and moisture changes should be placed near the bottom of the slab. The stresses set up by a change in temperature are insignificant, except in arid regions, compared with those set up by moisture changes, and ordinarily reinforcement to provide for temperature stresses is unnecessary.

(38) What is the maximum permissible length of reinforced slab?

Your Committee can give no definite answer to this question. The slab length may be anywhere from 20 ft. to continuous, depending upon the many elements entering into the problem.

(39) What is the maximum permissible width of reinforced slab?

The widest pavement without cracks within our knowledge is 40 ft. It is probable wider pavement can be constructed.

(40) May the slab be made thinner if reinforcement is used?

It would appear that if properly designed a thinner slab may be used if reinforced. Definite information, however, is lacking to exactly determine this fact.

F—THE EFFECT OF JOINTS ON CRACKING

(42) What is the purpose of joints?

The Committee has no evidence of the failure of slabs in concrete roads due to buckling, crushing or spalling, and, therefore, consider the joint as primarily for the purpose of taking care of contraction.

(43) What is the objection to the construction of a concrete road without joints?

If a concrete road is constructed without considering the affecting factors, with the idea that nature will provide cracks, it will crack, but before doing so it will cause a weakening of the concrete for some distance on each side, which may result in a partial disintegration of the road. This is evidenced by short forked cracks appearing running off from the main crack. The cracks thus formed will gradually lengthen with time and will not only be irregular in alignment, but they will not be vertical.

(45) What is the proper width of joint?

A joint should be constructed as a contraction joint with little or no room provided for expansion, as all stresses in the concrete caused by expansion are compressive and may be absorbed by the concrete later, upon contraction, there will be little movement, and, therefore, only small tensile stresses will be induced and there will be less liability of cracking.

(48) What should be the position of the joint relative to the length of the road?

There are no available data to show the difference in effect of diagonal and square joints on expansion and contraction. Any joint, however, should be perpendicular, so as to give proper bearing between adjoining slabs when they expand.

(49) Is it necessary to provide a longitudinal joint between road and adjoining structure?

It is if the adjoining structure is rough and will cause longitudinal restraint, or if the abutting structure will not offer sufficient resistance to prevent being pushed out of place.

(50) Should joints be placed at street intersections and on curves in a country highway?

Joints should be placed at street intersections and at sharp curves, so as to allow of free movement, wherever the concrete is restrained.

G—THE EFFECT OF CHARACTER AND CONDITION OF SUB-BASE ON CRACKING

(52) Is it desirable to have the sub-base impervious?

The sub-base should be very dense, but not entirely impervious. Where a rich mixture is used in the construction of a road it would tend to dry out unduly, causing excessive contraction. If a smooth subgrade could be provided so that the friction would be reduced to a minimum it might be desirable to have an impervious sub-base, but this condition is very difficult to obtain.

(53) Is it desirable to have the sub-base thoroughly compacted?

It is desirable to have the subbase uniformly and thoroughly compacted so as to prevent uneven settlement.

(54) Is it desirable to have the sub-base flat, curved or dished?

The sub-base should be dished.

In conclusion, your Committee believes that it has been fully established that the change in moisture content of the concrete is of much greater moment than the change in temperature under normal exposure; and that a proper combination of change in temperature, change in moisture content and friction have made possible the construction of long slabs which have remained free from cracks; while under an unfavorable combination of these factors, which is to be found in certain localities, long slabs have invariably cracked.

With a proper understanding of the physical phenomena, which are recommended for investigation, and with a proper application of engineering principles by competent highway engineers, it is believed by your Committee that all expansion and contraction of concrete in roads can be so controlled as to permanently avoid cracking. R. J. WIG, N. H. TUNNICLIFF, W. A. McINTYRE, Committee.

Making Concrete Waterproof—"We found on analysis that there is not enough fine sand in the torpedo sand that we get commercially and we are adding about one part of plastering sand to five parts of torpedo sand in order to get the proper grade of sand for watertight concrete. We are not putting in any waterproof material at all and are relying on the ingredients of the concrete. We were pursued, in the beginning of the work, by every waterproofing representative in town, who wanted his particular process specified. . . . I refer particularly to work in connection with filter plants and the like where they are building tanks 15 or 20 ft. high which must stand water pressure and not seep into a room that is used for working purposes. One part plastering sand to five parts torpedo sand is as near as I can find the way nature mixes it in the pit, but it is not the way it is usually specified."—Langdon Pearse, "Journal of the Western Society of Engineers," Dec. 1913, in discussing filter-plant work.

Improvements on the Louisville & Nashville R. R.

SYNOPSIS—The carrying out of extensive improvement work on the Louisville & Nashville R.R. System was noted in our issue of Feb. 5, 1913, in connection with an article describing the improvements (both by contract and by company-force system) on the line between Nashville and Birmingham. The present article deals with the improvements of the line from Cincinnati to Atlanta, and with the construction and reconstruction work for a new line into the Kentucky coal-field district.

As explained in the former article, the preparation of plans is done under the Chief Engineer, W. H. Courtenay, while the letting of contracts and the supervision of construction work is under the Chief Engineer of Construction, John H. Peyton. On the work described below, the division engineers are H. G. Laird for the line between Covington and Winchester, Ky., and E. D. Sloan for that between Winchester and Jackson, Ky.

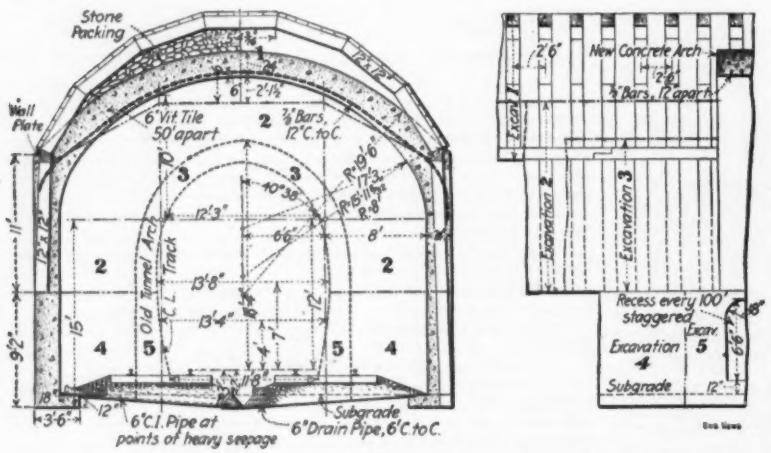


FIG. 1. ENLARGING A SINGLE-TRACK TUNNEL TO CARRY TWO TRACKS, AT COVINGTON, KY.; LOUISVILLE & NASHVILLE R.R.

The Cincinnati-Atlanta line of the Louisville & Nashville R.R. is composed largely of roads built by several small companies and gradually acquired and connected by the L. & N. R.R. Under such circumstances, as is natural to expect, the location and construction of the completed through line were not such as would be required for efficient or economical operation. For two or three years improvement work has been in progress on the northern part of the line, and is now completed from Covington (opposite Cincinnati) to Paris, 80 mi., while the next stretch of 16 mi. to Winchester is well advanced. Beyond this the line is single track for 55 mi. to Sinks, and then another stretch of 34.3 mi. of double track is completed to Corbin. The work includes double-tracking, grade reduction and re-alignment.

The Louisville & Nashville R.R. uses both its own terminal station and the Union Station in Cincinnati, the two lines crossing the Ohio River on separate bridges and meeting at Latonia (5 mi.). Here two main lines diverge also, one for Louisville, Nashville and New Orleans; and the other for Atlanta. At De Coursey, near Latonia, therefore, there are extensive freight yards for these two main lines.

The road from Covington south to Winchester was built originally by the Kentucky Central R.R., and as it was of rather light construction, with steep grades and circuitous alignment to avoid heavy work, it was not well adapted for the main-line traffic which it now carries. Further, the traffic between the points mentioned will be increased materially with the development of the eastern Kentucky coal fields and the completion of the new line from Winchester to this district, which is described later.

IMPROVEMENTS BETWEEN COVINGTON AND PARIS

Between Covington and Paris there is no great change in the alignment or grade, as the location is on the only practicable route for a low-grade line, through the range of hills, and follows along the Licking River for the greater part of the distance. The principal work has been in providing a second track, but at the same time the profile has been improved by lowering summits and raising sags. The second track also in some places swerves from the original line in order to secure more favorable alignment or grade.

The maximum grades now are 0.3% against northbound traffic (or 0.4% south of Winchester) and 0.5% against southbound traffic. The sharpest curves are 6°.

Three tunnels existed on this line. At two of these, the new second track was diverted far enough to allow of driving a second single-track tunnel in one case and an open cut in the other without affecting the old tunnel. The new tunnel is 2096 ft. long. At Covington, however, the work was restricted to the old right-of-way through the city, and here the tunnel, which is 751 ft. long, was enlarged, as described below. All the tunnels are lined with concrete.

There are no large bridges, although the line crosses the Licking River several times between Latonia and Paris, on girder spans.

The freight yard at De Coursey is being enlarged and rearranged to accommodate the increasing traffic. At Paris, the present yard will be enlarged, being used largely as an interchange yard for the traffic of the general divisions meeting at this point.

Relief or passing tracks are provided at intervals. In some cases they are outside of the main double-track line, in which case they are usually in pairs (on opposite sides), with their switch connections overlapping about 100 ft. In other cases, the main tracks are spread to admit a single relief track between them (with Y-connections at each end) so that this track can be used for trains in both directions. All new passing sidings are being made long enough to accommodate 100-ton trains.

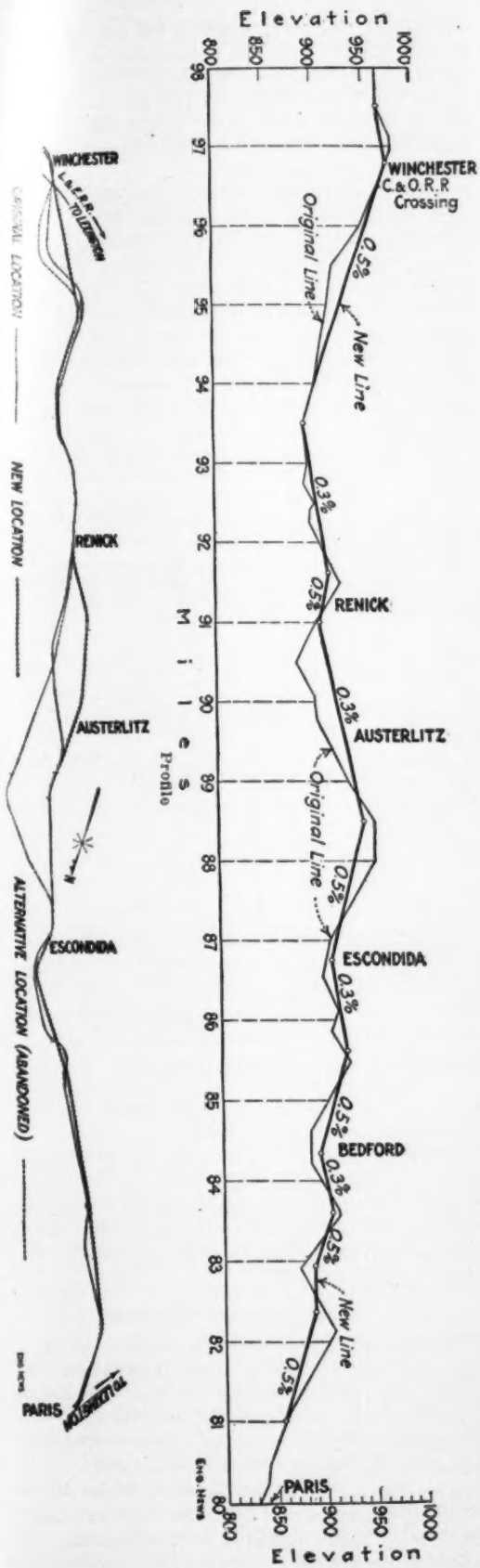


Fig. 2. Plan and Profile of Original and Revised Line between Paris and Winchester, Ky.; Louisville & Nashville R.R.

Double-Tracking a Tunnel

In double-tracking the line through Covington, it was necessary to keep to the original right-of-way, which necessitated enlarging the old tunnel and keeping it open for the very heavy traffic which it carries. This was work of an unusual and difficult character. The center line remains the same, and the plan adopted was to excavate around the old brick lining and build the new side walls and arch without disturbing this old masonry, which became a shield for the protection of trains.

The tunnel is 751 ft. long, and was completed in 1852. The material through which it is driven consists mainly of horizontal strata of limestone and shale. The old tunnel was of a narrow horseshoe section, with a maximum width of 13 ft. 8 in. (below which the sides sloped inward), and a height of 16 ft. 8 in. above the rails. It was lined with brick. The new tunnel is 29 ft. wide between vertical walls with a central height of 22 ft. above the rails. The roof has a radius of 16 ft., with haunches of 8-ft. radius. The concrete walls and arch are 2 ft. thick, the latter reinforced with transverse 7/8-in. steel bars near the intrados.

In the first place, a top heading about 7 ft. high was driven on the center line, with its floor about 2 ft. below the level of the intrados of the new roof arch and 3 ft. above the extrados of the old arch, as shown in Fig. 1. This was well timbered, and then lateral drifts (also timbered) were run downward to reach the position for the wall plates of the roof arch ribs. The material between these drifts was then removed, and the arch ribs erected. The excavation was then carried down to the crown and springing line of the old single-track arch; the wall plates of the double-track roof timbering were supported on posts, longer posts being set as the work progressed. The muck was hauled out in small cars.

The old masonry was then drilled from above, and blasted with light charges during available intervals between trains, the débris falling into cars standing on the tunnel track. This removed the masonry to the springing line. A working platform was then built above the clearance line for the use of the men in setting the forms and connecting the new tunnel arch. Beneath this platform was a smoke shield of tongued-and-grooved plank to protect workmen from the blast of the locomotives

Then the excavation was continued on each side, leaving the old side walls in place, the removal of these being the last stage. The forms for the side walls were then set and concrete deposited, and finally the arch forms were set and concreted.

Double-Tracking and Line and Grade Revision, Paris to Winchester

On this section of the line, the original location included numerous curves and a rather broken profile with grades of 1%, in order to follow the contour of the country and avoid heavy grading. The new alignment consists of long tangents and easy curves, with a maximum grade of 0.3% opposed to northbound trains and 0.5% opposed to southbound trains, as shown in Fig. 2. As it follows the general direction of the original line it intersects the latter at several points, and is sometimes above and sometimes below the old grade. This introduces some difficulties in carrying on the work without interfering with traffic, and in connecting up stretches of new work which are intersected by the old line.

The double tracking extends about a mile beyond Winchester, so as to include the junction with the new line to the coal fields (noted below) and also the widening and deepening of a heavy rock cut south of the town. At present the Lexington & Eastern division crosses above the main line, but the change in grade of the latter will make this a grade crossing, with connecting tracks for the interchange of traffic.

The track will be laid with 90-lb. rails on oak ties and 12 in. of stone ballast. All tracklaying is done by hand. For double tracking the existing parts of the line, the material is distributed from work trains. On the new portions, it is distributed on push cars. All tracklaying is done by company force and is under the direction of W. E. Smith, Superintendent of Construction.

Construction Methods

Of the three contractors on this Paris-Winchester work, the Hough & Spradlin Co. uses narrow-gage equipment; Duane & Carland use standard-gage equipment with 12-yd. and 16-yd. dump cars, and spreaders; the Mason-Hanger Co. uses both standard and narrow-gage equipment on different parts of its work. For the lighter work, different methods are used: drag scrapers at some points, dump carts at others, and small end-dump cars with mules at other points. The cuts require considerable blasting, as a large proportion of the material is limestone, but as the rock is loose and broken it shoots easily. The sides of cuts stand almost vertical, although soft shaly seams weather gradually. Portable deep-hole drilling rigs are used in the cuts. Fills are made usually by dumping from pole trestles.

There are some heavy rock cuts on the work, some of which are on the same alignment as the present track. Where the new cut includes the present line, it will be completed to a sufficient width for a running track, and when traffic is diverted to this, the old track will be torn up and the cut widened. As the rock stands fairly well (though somewhat shattered and containing thin shaly seams that weather), the new cut can be [illegible] very close to the old line. In some cases, as at [illegible], where the cut is deep and made in two lifts, the tra[illegible] shifted to the first bench (about 10 ft.) and then a[illegible] shifted to the final grade while the second cut is [illegible] made. Near Paris, the rock is being used to fill a lar[illegible] area for the site of a future yard.

It will be seen from the plan that the new line intersects the old line at several points. Where the grade of the former is 17 ft. or more above the latter, a temporary trestle will be put in to give an opening for the operated line. When work is sufficiently advanced, traffic will be diverted to a section of the new line, the old track abandoned, and the fill completed across it by dumping from the sides of the trestle. Where the difference in elevation is only 5 or 6 ft., a temporary connection or runoff will be built to connect the new and old lines, while the new fill is carried across the old line.

Colored labor is employed almost exclusively. In winter, however, there are many Austrians, but in summer the climate is too hot for them.

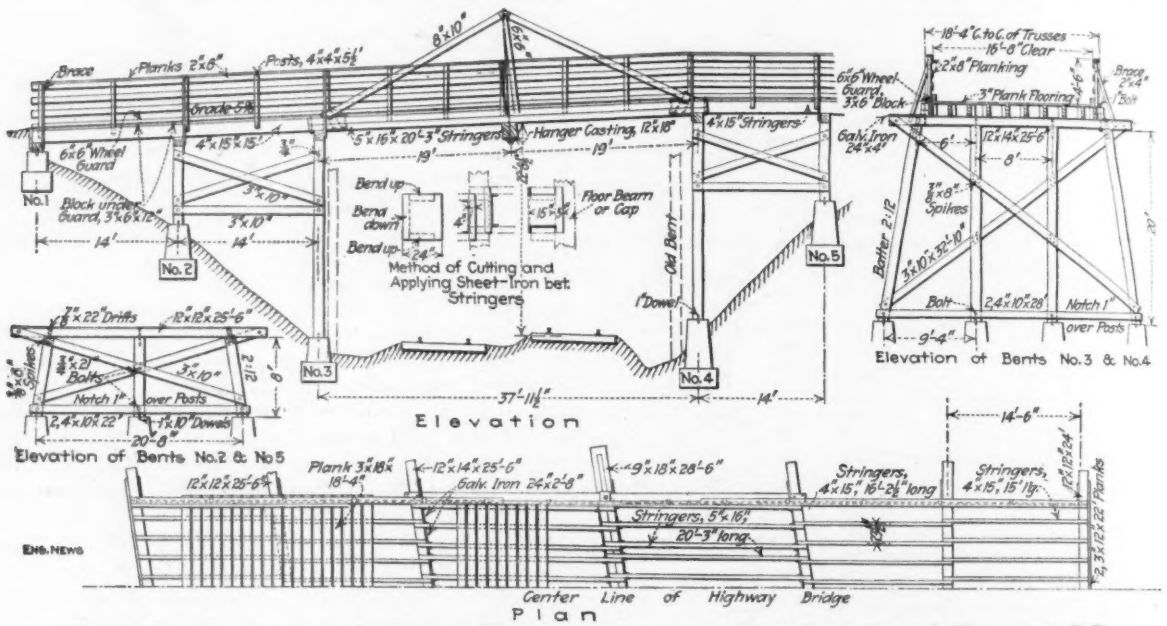

Elevation
Method of Cutting and Applying Sheet-Iron between Stringers
Elevation of Bents No.2 & No.5
Elevation of Bents No.3 & No.4
Plan
Center Line of Highway Bridge
ENG. NEWS

Fig. 3. Standard Design for Timber Highway Bridges over Cuts; Louisville & Nashville R.R.

Where contract work is in progress which involves the use of the old line by work trains, the railway company installs a temporary telegraph station at the track connection, with an operator who keeps the contractor advised as to train movements and sees that the line is kept clear for regular traffic.

Bridges and Culverts

The structures are mainly reinforced-concrete arch culverts and subways, but some roads are crossed by girder spans on abutments. In one case where the headway over a road required a minimum depth of floor, the 15-ft. span is composed of 19-ft. creosoted timber stringers 5x18 in., placed ½ in. apart. These spaces allow for drainage but are too narrow to let the 2-in. ballast fall through. The largest arch culvert has a span of 80 ft. and a rise of 17 ft. 9 in., with a radius of 54 ft. The ends of the arch spring directly from skewback seats cut in the solid rock,

just below the level of the bed of the creek. The thickness of the arch is 3 ft. 6 in. at the crown and 7 ft. 3 in. at the ends, and it has no steel reinforcement. The parapet walls are 9 in. thick, with triangular buttresses extending to the arch, and the wing walls are of similar design, 8 in. thick. All these walls have steel reinforcement.

One of the features of the new work is the elimination of road crossings at grade, highways being carried under the line by concrete subways or over the line by wooden bridges. In these latter structures, a bent on each side carries a triangular truss span over the tracks, and intermediate bents carry the approaches. These bridges are of creosoted timber.

The standard design for timber highway bridges over the tracks is shown in Fig. 3. The triangular truss span is 38 ft. long, the three chord members being 8x10 in., and the vertical 6x6 in. At the apex of the truss is an iron block with a pin for the hanger rod which carries a casting upon which rests the end of a transverse beam. The floor stringers are in lengths of 20 ft. 3 in., the ends resting on the caps of the bents and this beam, as shown by the plan. The stringers are 5x16 in., spaced 15½ to 23½ in. c. to c., with a floor composed of a single layer of 3x8-in. planks laid ¼ in. apart.

All timber is yellow-pine creosoted with 12 lb. of creosote per cu.ft., except that untreated timber is used for the floor and the hand railing. Wherever creosoted timber is used cut or bored in the field, the new surfaces are mopped thoroughly with hot creosote oil. Galvanized-iron sheeting is laid over the caps, and vertically between the ends of the stringer at the middle and ends of the main span, so that sparks cannot lodge on the caps. The dead load of stringers and floor is about 30 lb. per sq.ft. The bridges are designed for a live-load of a 12-ton roller or 100 lb. per sq.ft. of roadway.

Slides

On the Kentucky lines there has been considerable trouble with slides. At the north end of one of the new single-track tunnels for double tracking the Cincinnati line (near Ryland) there is a somewhat extensive slide, which has been in motion for some time, the upper stratum of earth sliding on a stratum of rock. The toe reaches nearly half way across the Licking River, and it will be necessary to clear out the channel. At another point on this same line a fill of loose rock has slipped and spread and continues to move. Adding new material on the top only adds to the weight and serves to continue the movement. The engineer in charge, Mr. Laird, proposes to borrow material near the toe, and build a fill about 15 ft. high and 40 ft. wide against the toe, thus affording resistance to the pressure.

On the new line to the coal fields (described below) there have been numerous slides along the Kentucky River, due largely to the presence of strata of soapstone in the coal measures. It is suggested that these may be checked by blasting along the toe so as to break up the slippery stratum upon which the overburden rests. In approaching Irvine there is a considerable stretch of side-hill cut along the Kentucky River, and so many slides developed (through cutting into the foot of the high slopes) that it was found necessary to relocate the line at a higher elevation. One of these extended so far into the river that the U. S. Engineers ordered its removal, and to avoid further trouble the alignment was changed.

Between Irvine and Beattyville, also, where the line follows the river and is at the base of similar steep slopes, numerous slides have occurred where the lowering of grade summits has resulted in cutting away the toe of the slope to a still greater depth. The material is mainly shale, with some soapstone seams, and it has been proposed to dynamite the toe of the slope so as to break up the smooth surface of stratum upon which the upper mass of material slides when a cut is made across it.

New Line to the Kentucky Coal Fields

The Louisville & Nashville R.R. has a line from its Cincinnati-Atlanta division (described above) extending to McRobert, in southeastern Kentucky (270 mi. from Cincinnati), and in view of important coal-mine developments near the end of this line (in Letcher County), extensive operating conditions and greater traffic capacity. The improved line will be single-track throughout.

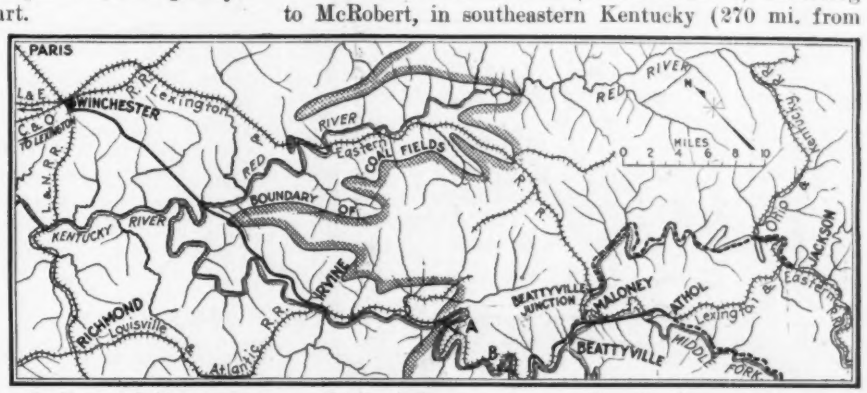


Fig. 4. Map of New Line and Improvements between Winchester and Jackson, Ky.; Louisville & Nashville R.R.

This work is shown on the map, Fig. 4. The present route is by the Lexington & Eastern R.R. (L. & N. R.R. System) from Winchester, on the main line, to Jackson, Ky. (73 mi.), and thence to McRobert, 175 mi. from Winchester. On the present L. & E. R.R. line to Jackson there are maximum grades of 1.5% and curves as sharp as 12°. The improvement consists in building a new 26-mi. low-grade line from Winchester to Irvine, a point on another branch of the L. & N. R.R., and then improving this branch along the Kentucky River from Irvine to Beattyville (34 mi.) and Jackson (56 mi.), making a total distance of 82 mi. This branch is the old Richmond, Nicholasville, Irvine & Beattyville R.R.; it is now known as the Louisville & Atlanta R.R., and is coal trains for the run to Cincinnati.

On the new line from Winchester to Irvine, the maximum grade is 0.4%, and the sharpest curves are 6°. East of Irvine, on the revised line, the maximum grade

will be 0.3% against loaded trains (northbound) and 0.5% against southbound trains.

Beyond Irvine there will be no material change for a considerable distance, the location following the steep side-hill ground close to the Kentucky River, and the topography is such that it affords little opportunity for changing the location. At two points (*A* and *B*), however, future cutoffs have been proposed to eliminate long loops following the windings of the river, but each of these would require two bridges over the river and heavy excavation and tunneling across the spurs. At a loop

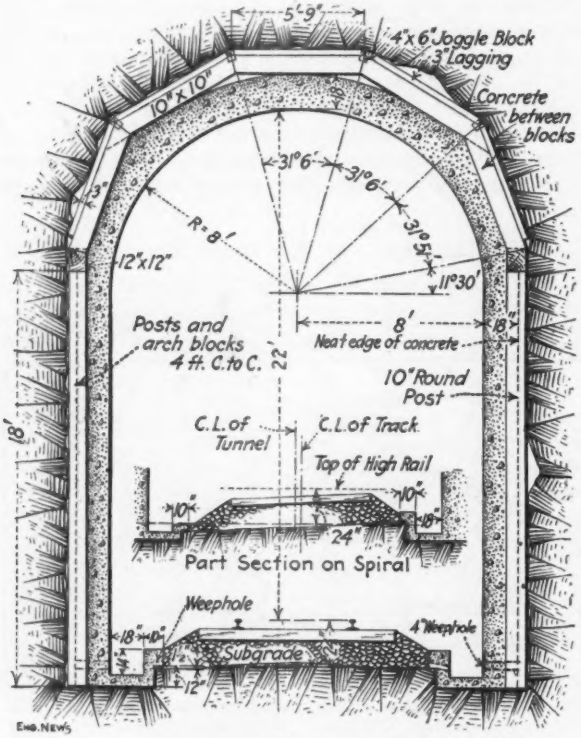


FIG. 5. STANDARD CROSS-SECTION FOR SINGLE-TRACK TUNNELS; LOUISVILLE & NASHVILLE R.R.

where Contrary Creek enters the river the alignment will be slightly changed and the grade reduced. A little beyond this, and just west of Beattyville, begins a 9-mile stretch of improvement, which includes a cutoff leaving the present line at Maloney, where it crosses to the south side of the river and makes a direct run, recrossing the river near Athol, and rejoining the present line at that place. This will eliminate the present indirect route by way of Beattyville Junction and also a winding stretch of line on the north side of the river.

The profile between Irvine and Beattyville originally included grades of 1.25% and 1.7%, but the profile will be improved materially, giving a maximum of 0.3% westbound and 0.5% eastbound. The improvement consists largely in lowering the summits in shale cuts, and there has been a good deal of trouble from slides in these sidehill cuts. The heaviest cut is 130 ft. deep on the center line, but the deepening of the cut will be cheaper than building a tunnel.

At Athol, the line leaves the Kentucky River, and from that point to Jackson the work consists mainly in slight improvements of alignment, and in relaying the track with 90-lb. rails on stone ballast. This work is being done by company force.

A large yard will be built at Irvine for making up the owned by the Louisville & Nashville R.R.

TUNNELS

There will be four tunnels; two on the new Winchester-Irvine line and two on the Beattyville-Athol cutoff. One of these latter will replace an existing tunnel in loose material which has given considerable trouble. The new one will be parallel with and at a short distance from the old one (with the center lines only 25 ft. apart) but at a lower elevation, where it will be in a solid formation. The longest tunnel has a length of 800 ft.

The tunnels are of the company's standard section, shown in Fig. 5, with a clear width of 16 ft. on tangents and a height of 22 ft. above the rail. The area of excavation is 495.02 sq.ft. The timbering consists of 10-in. round posts carrying the wall plates for the arch blocks, upon which is 3-in. lagging. The posts and blocks are spaced 4 ft. c. to c. All spaces outside the timbering are packed with hard, thin ledges of limestone, including the space between the lagging and the concrete. The concrete lining is 18 in. thick. An interesting detail is the concrete ditch, with curb wall to retain the ballast.

The quantities per lin.ft. of tunnel are as follows: (1) excavation, 18.33 cu.yd.; (2) concrete, 3.54 cu.yd. with posts 10 in. diameter or 3.48 cu.yd. with posts 10x10 in.; (3) timber, 206 or 243.5 ft. b.m., with posts as above. The end of one tunnel is on the spiral of a 4° curve, and the half-width (and roof radius) increases gradually from 8 ft. on the tangent to 8 ft. 5½ in. on the curve, while the offset between center lines of tunnel and track reaches a maximum of 0.479 ft. at the P. C. C. The floor of the tunnel is also dressed to a slope, as shown

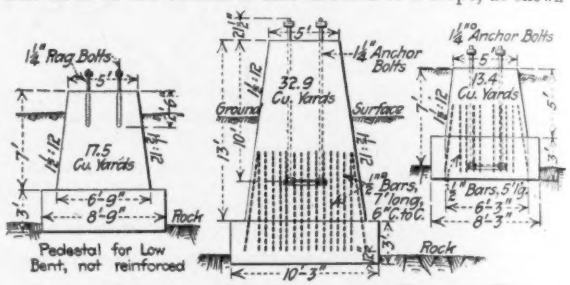


FIG. 6. CONCRETE PEDESTALS FOR VIADUCT BENTS (HOWARD CREEK VIADUCT); LOUISVILLE & NASHVILLE R.R.

(The vertical bars are for reinforcement against stress due to wind pressure.)

in the smaller cross-section, to facilitate giving the track the requisite superelevation.

BRIDGES AND CULVERTS

There will be five steel viaducts on the new line to Irvine. These will be 350 to 1800 ft. long, the highest (at the Red River) having its rails 150 ft. above low-water level. This has 40-ft. tower spans with 60- and 80-ft. intermediate spans; and a central deck-truss span of 200 ft. carried by towers with triangular bents. There will be very few truss spans on the line.

The numerous ordinary trestles on the old lines are being filled, but the Lexington & Eastern has some creosoted pile trestles with ballasted floors and these are being left in place. There are some road crossings

formed by creosoted stringers laid about 6 in. apart and having a plank and ballasted floor.

The typical design of pedestals for the viaduct towers is shown in Fig. 6, showing pedestals for the Howard Creek viaduct between Winchester and Irvine. This is 1200 ft. long and 170 ft. above the water. A rather unusual feature is the use of vertical reinforcing bars, the purpose of which is to resist tensile stress due to horizontal wind pressure. These are ½-in. twisted square bars spaced 6 in. apart and 3 in. from the surface on all four sides. All pedestals are square, with exposed edges rounded or chamfered, and all are carried down to rock foundation. Particular care is taken in locating and securing the anchor bolts, and they are required to be set within $\frac{1}{16}$ in. of their true position. Heavy timber templates are used to prevent displacement of the bolts when depositing concrete, the templates being accurately bored and set and securely braced. At least three check measurements on the position of each group of anchor bolts are made before the concreting is finished, the third measurement being made when the concrete is within 2 ft. of the top of the pedestal.

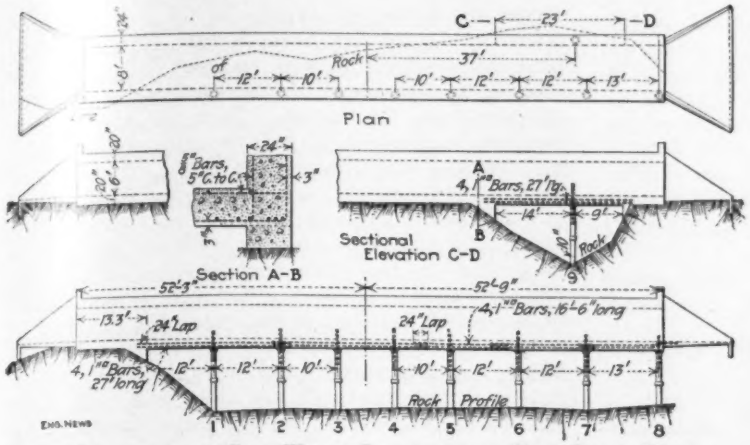


FIG. 7. CULVERT WITH SIDE WALLS SUPPORTED ON PIPE POSTS FILLED WITH CONCRETE; LOUISVILLE & NASHVILLE R.R.

Ordinary cast-iron pipe in 12-ft. lengths is used for drainage openings under embankments over 20 ft. high, while cast-iron lock-joint culvert pipe in 3-ft. lengths is used under banks of less height. Rectangular culverts of concrete are of various sizes, from a minimum of 4x4 ft. to a maximum span of 16 ft. Concrete arches for roads and waterways are used from a minimum of 10-ft. span up to 50 ft. and even 80 ft. Plain concrete is used for arches up to 24-ft. span (and for larger spans springing directly from the rock), while the wings and parapet walls are of reinforced concrete. Foundations are required to be capable of sustaining a pressure of 4000 lb. per sq.ft. without settlement. All reinforcement is of cold-twisted steel bars, and all U-bars or stirrups have a 3-in. hook at the end of each leg. Where a road subway has a concrete floor, paving must be placed and maintained on this to protect the concrete from wear.

CULVERT ON CONCRETE POSTS—In one case where a culvert was supposed to have a rock foundation, a sharp curve was found in the rock so that part of the middle of the culvert was over soft material. The position could

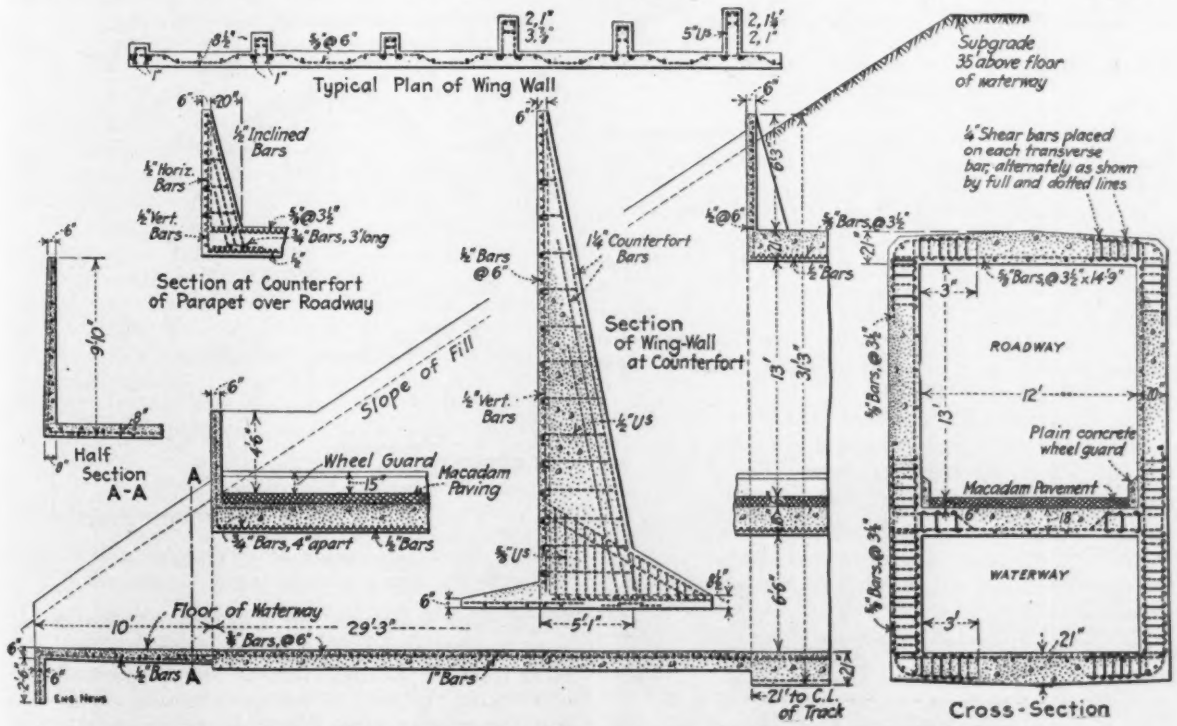


FIG. 8. DOUBLE-DECK CULVERT FOR HIGHWAY AND STREAM; LOUISVILLE R.R.

not be changed as the contractor was ready to build the fill. A method of supporting this part of the culvert on concrete posts was proposed by H. H. Cartwright, Assistant Engineer, and carried into effect.

Along the lines of the side walls, 10-in. and 12-in. well casing was driven to a solid foundation, and the silt pumped out. Then the pipes were filled with concrete, and when this had set the culvert was built, its floor being heavily reinforced. Under one wall there are eight of these posts, 10 to 13 ft. apart. Under the other wall only one post was required. Steel bars inserted in the pipes project upward into the side walls.

Soundings were taken to ascertain the depth to rock.

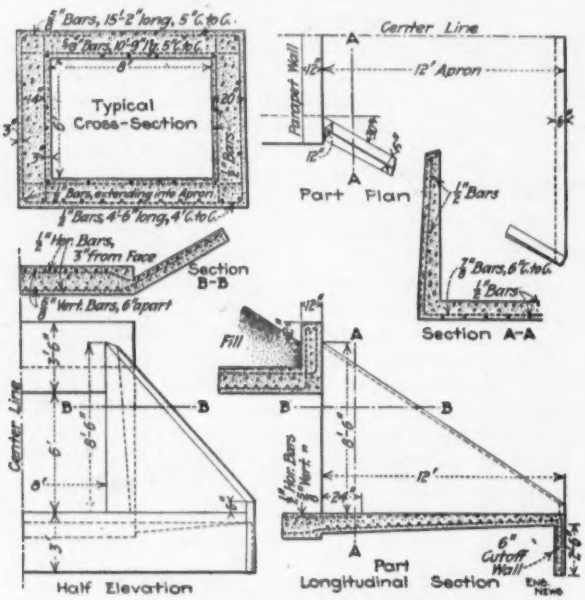


FIG. 9. STANDARD TYPE OF BOX CULVERT, LOUISVILLE & NASHVILLE R.R.

Each pipe was driven in two lengths for convenience in handling and an improvised wooden piledriver was used to drive the pipe. This construction is shown in Fig. 7.

DOUBLE-DECK SUBWAY—Another special form of under crossing is a double-deck concrete subway of the box type. The lower compartment is 12x6½ ft., and carries a creek, while the upper compartment is 12x13 ft. and carries a highway. The former is considerably longer than the latter, the road diverging from the line of the culvert as soon as it has passed under the fill. This structure is shown in Fig. 8.

BOX CULVERTS—The standard type of reinforced-concrete box culvert or underpass is a box culvert 10x11½ ft., with a 6-in. macadam pavement. Fig. 9 shows the standard 8x6-ft. culvert, used with a depth of cover of not less than 3½ ft. The quantities for this latter are as follows:

Fill (over roof), ft.	3½ to 20	20 to 30	30 to 40
Side walls, in.	17	20	24
Floor and roof, in.	15	17	20
Masonry, per lin.ft. culvert, cu.yd.	1.63	1.93	2.37
Masonry, parapets (2), cu.yd.	1.80	1.75	1.63
Masonry: wings (both ends), cu.yd.	18.57	18.57	18.57
Steel, per lin.ft. culvert, lb.	202.94	247.65	263.06
Steel in parapets (2), lb.	54.00	53.30	52.60
Steel in wings (both ends), lb.	2742.50	2472.50	2472.50

ARCHED CULVERTS—The standard type of arch culvert used on rock foundations is shown in Fig. 10, which represents the design for 20-ft. span, with a minimum and maximum cover of 3 ft. and 15 ft. from the grades to subgrade. The concrete for such structures is a 1:2½:5 mixture.

CONSTRUCTION WORK ON THE WINCHESTER & IRVINE LINE

The contract for the new low-grade line between Winchester and Irvine was awarded by the L. & N. R.R. to Winston & Co., of New York, in June, 1912. This contract covers all of the work necessary to complete the roadbed to subgrade, with the exception of the steel superstructures for viaducts. It is for a single-track line with roadbed 24 ft. wide in cuts. The length of the new road is a little over 26 miles. The maximum grade is 0.4% opposed to westbound and 0.5% opposed to eastbound trains, with short and easy curves. The general direction of the road is north and south, while the general direction of the drainage is from east to west. For particulars of the construction work we are indebted to Roswell D. Trimble, of the contracting company.

The total amount of open-cut excavation on this work will be about 2,000,000 cu.yd., of which about 60% is solid rock, 20% loose rock, and about 20% earth. There are two tunnels through thin stratified limestone; one is 550 ft. and the other 670 ft. long. The concrete work amounts to about 22,000 cu.yd.

Owing to the very heavy work required and a time limit for completion in 18 months, it was necessary that a large number of shovels should be placed on this work. As all of the heavy work was on the interior sections, with a very rough country between these sections and existing railway lines, the problem of placing shovels on the work was very serious, and caused some contractors to hesitate about undertaking the work at a close figure.

On going over the work, preparatory to making a bid,

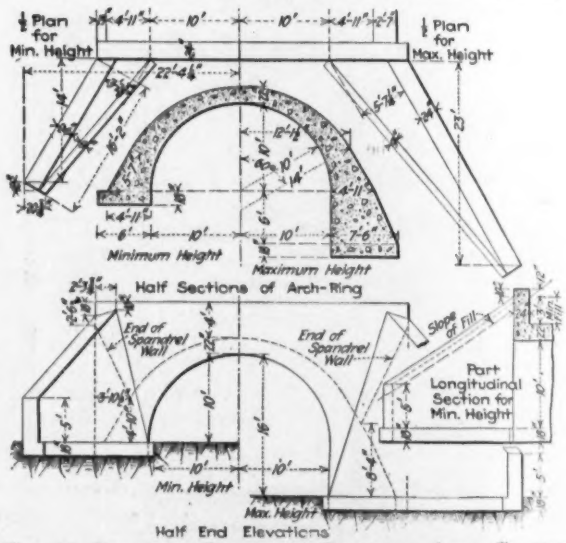


FIG. 10. STANDARD DESIGN OF CONCRETE ARCH CULVERT OF 20-FT. SPAN; LOUISVILLE & NASHVILLE R.R.

Mr. Trimble found that for a distance of about 15 mi. from the Irvine end, the line was located not over three miles from the Kentucky River. He conceived the idea of utilizing the river to transport shovels and plant to points nearest to where it was desired to operate them. On the contract being awarded to his company, he remod-

eled a shallow-draft lighter for this purpose and built a terminal for loading at Ford, Ky., where the Kentucky Central Division of the L. & N. R.R. crosses the Kentucky River. By this method, 11 of the shovel plants were placed on the work. Nine of these were 65- and 70-ton machines, and two were 45-ton machines. Six of the plants had standard-gage train equipment, and five had narrow-gage equipment.

Due to the care in making preparations for transporting by water and providing in advance for all possible contingencies this work was accomplished successfully without any accident or loss. The movement was made in very much less time and at very considerably less cost than would have been possible had the outfits been moved across country. The best record was two weeks for placing (at cut ready to start work) a shovel plant, including trains and other equipment. Had this outfit been moved across country the distance would have been 16 miles over hilly country.

Mr. Trimble states that when the intention to move shovels on lighters was made known, it was claimed that this was too risky and would be a failure. The work was undertaken in the face of such discouraging opinions, "and now everyone says of course it was the right way to place these shovels on the work."

Midway between Winchester and Irvine the road crosses the Red River on a steel viaduct 1780 ft. long and 190 ft. high from bed of river to base of rail. There will be four other viaducts from 360 to 1235 ft. long, 90 to 110 ft. high to base of rail.

From Winchester to the Red River the rock is a limestone formation. From the Red River to Irvine the rock is a hard bastard sandstone in heavy ledges and boulder formation. Considerable hard flint is embedded in this rock, making it difficult to drill. It breaks up in heavy masses, requiring constant mud-capping and block-holing to reduce it to a size that steam shovels can handle. Underlying this rock is soapstone to a considerable depth.

The work has required an excessive use of explosives, and 40% nitro-glycerin dynamite has been used exclusively, except in tunnels. Even where blasting powder could ordinarily be used to advantage, particularly in the soapstone, it was not practicable to use it here on account of the holes being too wet.

There are no very large cuts, the largest being about 135,000 cu.yd., but many contain from 20,000 to 80,000 cu.yd. The cuts are comparatively short, but have steep approaches, making it difficult to excavate the top lifts with shovels. The deepest open cut is 108 ft. in depth. The ordinary fills are made generally by dumping from temporary trestles. These have two-post bents of round timbers or poles, with log caps and stringers and a pair of pole braces from the top of each bent to the foot of the adjacent bent.

With few exceptions the fills are not long, but deep; many of them being from 50 to 90 ft. in depth. Owing to this depth, it was impossible to use trestles for dumping material from trains, and three aërial dumps or cable dumps of 400- and 600-ft. span have been used with satisfactory results. These cable dumping bridges were described and illustrated in our issue of Nov. 20, 1913.

Owing to the height and abrupt nature of the ground at most of the cuts, it has been very difficult to operate trains for excavating the top lifts. Hoists have been used for pulling up and lowering trains; also locomotives operated on down grades at the further side of the cut and connected with train by cable. Another method was to cut in shovels on the lower lift and spot the cars. All of these methods have been slow, but the best results have been obtained by locomotives with cables.

The work is being handled with sub-contractors, and it is expected to complete the contract this year.

The Removal of Iron and Manganese from Water

The perplexing problems associated with the removal of iron, or iron and manganese, from underground water-supplies were clearly explained, some interesting recent experiments in 1913 at Cohasset, Middleboro and Brookline, Mass., were described and deferrization plants built for Cohasset and Middleboro in 1913 were described in a paper by Robert Spurr Weston, of Boston, Mass., read before the New England Water Works Association on Feb. 11, 1914. The experiments were conducted and the plants designed by Mr. Weston. The somewhat lengthy paper, presumably with illustrations, will appear in the *Journal* of the Association. We give here an abstract of the paper and include practically all of Mr. Weston's description of the plants designed to treat the water-supplies of Cohasset and Middleboro.

In opening his paper, Mr. Weston stated that since Charlottenburg, Germany, in 1874, freed its water-supply from iron by aëration and filtration, the practice has been adopted by many other cities, especially in Germany. Besides Charlottenburg the German cities include Berlin, Hamburg, Breslau, Dresden, Leipsic, Kiel, Brunswick, Hanover and others. In the United States iron-removal plants had been built, prior to 1913, at Reading and Marblehead, Mass., Far Rockaway, N. Y., several places on the New Jersey Coast [including Asbury Park, Atlantic Highlands and Keyport.—Ed.], Garretsville, Ohio, West Superior, Wis., Richmond, Mo., and a few other places.*

The Problem of Deferrization

The iron-removal problem, in its simplest form, can be solved by (1) aëration to oxidize "the iron from the soluble unoxidized form to the insoluble oxidized form, or from ferrous to ferric hydrate," and (2) by filtration to remove the precipitate. Difficulties arise with precipitation (not with oxidation) when "interfering substances" are present in sufficient amounts to "prevent the precipitation of the iron, holding it in the semi-soluble or colloidal form, and making its removal by filtration difficult. Three causes of interference explained by Mr. Weston are carbon dioxide, organic matter and manganese.

Carbon Dioxide—Like other acids, carbon dioxide interferes with the precipitation of ferric hydrate. It must, therefore, be reduced as much as is practicable where iron is to be removed. In the case of waters of peculiar organic composition, as those from beneath alluvial or marshy lands, "aëration can be carried so far as to make it impossible to coagulate the oxidized iron before filtration," as witness Reading, Mass., and West Superior, Wis.

*Some of these plants have been described in our columns as follows: Asbury Park, Atlantic Highlands and Keyport, June 6, 1896; Reading, Mass., Nov. 26, 1896; Far Rockaway, N. Y., Apr. 12, 1900; West Superior, Wis., Feb. 21, 1901.—Ed.

ORGANIC MATTER—Some kinds of organic matter assist precipitation of iron and the iron acts likewise on the organic matter. Other kinds hinder, apparently all the more when relatively fresh. As with carbon in acid, so with organic matter, particularly "new matter or that rich with humic acid," aëration may be carried too far. In such cases the use of the usual form af aërator may be objectionable; or at least it may be preferable to give the water what may be termed contact treatment by passing it through broken stone or coarse coke, submerged.

MANGANESE—Little attention was paid to manganese until 1906 when this substance rose to 220 parts per million in the well water supplied to Breslau [see ENG. NEWS, Oct. 4, 1906]. Manganese in water-supplies has rarely been reported in the United States. The substance is closely related to iron, chemically, but "reacts and precipitates more slowly, and it possesses the power of preventing the removal of the last traces of iron" unless the manganese be removed with the iron.

DEFERRIZATION EXPERIMENTS IN 1913

During 1913, Mr. Weston conducted iron-removal experiments for the Cohasset Water Co. and the towns of Middleboro and Brookline, all in Massachusetts. He submits the accompanying table of analyses of these waters, and comments on them as follows:

ANALYSES OF WELL WATER FROM COHASSET, BROOKLINE AND MIDDLEBORO, MASS.
(Parts per million)

Source of sample	Cohasset Well	Brookline Wells Averages	Middleboro Well
Turbidity—silica standard ..	3.0	4.0	11.0
Color—platinum standard ...	44.0	29.0	41.0
Oxygen consumed	5.6	1.88	0.65
Nitrogen as: Free ammonia..	0.052	0.061	0.049
Albuminoid ammonia—total	0.097	0.088	0.046
Nitrites	0.000	0.001	0.001
Nitrates	0.04	0.19	0.42
Chlorine	14.3	8.5	6.6
Alkalinity	30.0	40.1	
Hardness by soap method....	16.0	47.1	26.6
Iron, Fe	0.73	0.60	1.49
Residue on evaporation: total	80.0	105.0	68.0
Manganese, Mn	0.23	0.26	0.75
Carbon dioxide (free and half bound)	49.82	31.8	45.0
Dissolved oxygen	1.58	1.32	3.02

The remarkable fact about the above waters is the similarity of many of their characteristics; they have about the same turbidity, color and free ammonia; they are all soft, contain little mineral matter and their chlorine content is what one would expect from their location. They differ in the amounts of oxygen consumed, albuminoid ammonia, nitrates, iron, manganese and carbon dioxide; it is these differences which are significant.

The Cohasset water is the most difficult to treat. Its chief interfering substance is organic matter, but in addition carbonic acid gives trouble. Both organic matter and manganese interfere at Brookline, but the difficulty here is not so much the interfering bodies as the small amount of iron—it is easier to precipitate five parts of iron than one part. At Middleboro, the chief interfering substance is manganese.

In the Cohasset experiments aëration, aëration and partly submerged stone, and filtration were tried, as well as some or all of these combined with (1) sulphate of alumina; (2) this and soda ash; (3) these two and bleaching powder; (4) bleaching powder alone and (5) with sulphate of alumina.

The Brookline experiments, conducted jointly by F. F. Forbes and Mr. Weston, were with aëration, coagulation and sand filtration, and with the same plus a coke trickler inserted between the aërator and the coagulation basin. Tricklers of 2,- 5- and 10-ft. depths were tried. The Brookline experiments had not been complete when Mr. Weston read his paper.

At Middleboro, experiments previously made by the Massachusetts State Board of Health with aëration and filtration were continued, with a stone trickler inserted between the two, and with consideration given to the manganese content of the water.

THE COHASSET PLANT

The first two Cohasset experiments were made by the Norwood Engineering Co., with the assistance of A. L. Gammage, of Mr. Weston's office. Later in 1913, plans for an iron-removal plant for Cohasset were made by the company named and a contract for its construction was let to that company. Water will pass from an aërator onto a trickler of coarse coke 10 ft. deep, designed to operate at a rate of 75,000,000 gal. per acre per day. The trickler will be piped to work submerged, partly submerged or nonsubmerged, but probably it will be operated partly submerged. From the trickler the water will pass to a small subsiding basin, which like the trickler is located beneath the aërator. The settled water will go to three standard mechanical filters and their effluent water will pass to a basin beneath the filters. At times but little aëration will be possible if the iron and color are removed, but the contact period must not then be reduced.

THE MIDDLEBORO PLANT

Mr. Weston's description of this plant may be quoted as follows:

The design of the Middleboro plant was made during the experiments and the plant was built during the summer of 1913. The contractor for the main structures was the Briggs Engineering & Construction Co., of Providence, R. I., and the low-lift pump was supplied by the Power Equipment Co., of Boston. The details of the design and construction were worked out and supervised by G. A. Sampson, with T. F. Dorsey as resident engineer. The plant consists of the following parts:

A De Laval steam-turbine-driven centrifugal pump, having a capacity of 1,000,000 gal. a day. This pump takes water from the present well and discharges it on top of a pile of coke—a so called trickler—through a number of sprinkling nozzles.

A trickler built of concrete, 30 ft. in diameter, outside measurements, and containing 10 ft. of coarse coke supported upon a concrete grillwork, above the true bottom. The water from the trickler falls upon its true bottom and flows through a pipe into the settling basin.

The settling basin is built of concrete and has a capacity of 40,000 gal. It is covered with a concrete roof and earth to prevent freezing.

From the settling basin the water flows into the two compartments of the filter, having a total area of 0.1 acre. These filters are simply concrete basins with groined roofs, covered like the settling basin with earth to prevent the freezing of the water in winter. On the bottom of the filter are laid tile underdrains covered with 12 in. of graded gravel, supporting in turn 3 ft. of medium-sized sand.

From the two filters, the water flows into a regulator house in which are located all of the valves and gages for operating and regulating the filters.

From the regulator house the water flows into a filtered water basin which holds 42,000 gal. This is large enough to enable the pumps and filters to operate regularly and to supply the hydrants during ordinary fires. For fire purposes this supply is at the rate of at least 1,000,000 gal. in 24 hours, or twice this amount during one hour. The present consumption is about 335,000 gal. per day, and the maximum consumption for any one day has been 873,000 gal.

AERATOR AND TRICKLER—The aërator consists of a system of piping connecting the discharge from the low-lift pump with 37 2-in. nipples. On each nipple is screwed a cap drilled with 24 $\frac{1}{16}$-in. holes and pressed to make the upper surface convex and the axes of the holes radial. These sprays discharge upward, discharging the water on the sur-

face of a layer of coarse coke, 10 ft. deep, contained in a concrete tower 28.3 ft. in diameter.

The coke is supported upon 2x6-in. reinforced-concrete beams spaced 2 in. apart and supported upon a berm, 6 in. wide, around the inside of the trickler wall and also upon four cross walls, 6 in. wide, resting on the floor.

The floor slopes to a channel leading to the 18-in. cast-iron pipe which connects with the subsiding basins and filters. A shear-gate provides for the closing of this outlet, when the trickler may be filled; then the valve may be opened quickly and the excess of accumulation in the coke flushed out through the subsiding basin and into the drain. It is not expected that flushing will be required oftener than once a year.

FILTERS AND APPURTENANCES—The two filters have no doors and runways, but special sand boxes have been designed, into which the sand removed by scraping is dumped and washed out into bins placed outside the filter. The sand used in the filters had an effective size of 0.31 mm. and a uniformity coefficient of 1.80. It was a natural bank sand which did not require washing.

The regulating devices are of the orifice type. Indicating rate- and loss-of-head gages are provided for each filter.

COST—The complete plant has cost about $18,000, including engineering.

RESULTS—The filters were started on Sept. 26, 1913. They had been raked twice and scraped once prior to Jan. 12, 1914. The yield for this first run was 40,000,000 gal., or 400,000,000 gal. per acre per day. The filters have been in operation so short a time that the effluent varies in quality considerably, particularly just before and after scraping or raking the filter. The average results are as follows:

AVERAGE RESULTS OF OPERATION, MIDDLEBORO; FIRST RUN, SEPT. 26, 1913, TO JAN. 12, 1914

(Parts per million)

Determination	Well	Settling basin effluent	Filter effluent
Color	48.0	22.0	5.0
Turbidity	5.0	3.0	1.0
Iron	1.62	0.46	0.17
Manganese	0.67	0.36	0.27
Hardness*	27.3	...	23.4
Oxygen consumed	1.79	1.53	1.05
Carbon dioxide*	41.0	4.2	4.6
Dissolved oxygen*	2.95	10.20	9.55

*One determination.

General Conclusions

The diversity of character shown by different waters and the consequent diversity of treatment which they may require, together with some remarks on the choice of a filter, are thus well put at the close of Mr. Weston's paper, as follows:

In this paper have been given the results of experience with three waters containing iron and manganese, one of which could not be aërated to saturation without causing some of the iron and color to escape removal, and one of which required complete oxidation to effect a satisfactory removal of manganese. The third water demands complete aëration and long contact as well, and contingencies may arise when this contact should be effected with the material submerged.

Other things being equal, it is well to remove the carbon dioxide as completely as practicable in order to prevent subsequent corrosive action of the water upon the distribution piping; consequently aëration and contact in a trickler should be employed as far as possible, and submerged contact when this method fails.

Naturally, the choice of methods depends upon the character of the untreated water, and if the writer has made it plain that not only do different supplies vary, but that even the water from the same well may require a varying treatment, he will feel repaid.

Regarding the choice of filter, it may be stated that properly treated water can be filtered either through slow or rapid filters, with good success. The high rates possible with a sand filter, and the infrequency of cleaning when the pretreatment is adequate, usually makes it more economical than the mechanical filter; furthermore, when manganese is present, it undoubtedly gives more uniform results.

Proposed Water-Works for Watervliet, N. Y., recommended by Solomon, Norcross & Keis, of Atlanta, Ga., include a low reinforced-concrete dam on the Normanskill, a water-power pumping plant, a wood-stave supply main, a service reservoir and a distribution system, at an estimated cost of $497,000—or $562,350 if cast iron were used for the supply main.

An Ice-Emergency Water Intake at Waukegan, Ill.

A trouble which has been experienced periodically in the water-supply of Waukegan, Ill., is the clogging of the intake (in Lake Michigan) by ice, and this led to the construction of an independent emergency intake, which has been put in service recently.

The city takes its water-supply through a 24-in. cast-iron pipe extending 4000 ft. into the lake. The depth at the intake is 28 ft., and the water enters a short piece of upright pipe which is connected to the intake pipe by means of a tee. The depth of water over the mouth of the pipe is 24 ft., and no intake structure can be built as the U. S. Government requires 26 ft. of water. Slush ice (and probably anchor ice occasionally) has caused a stoppage of the mouth of the intake for an average of ten times each winter, the period of stoppage being from ½ hour to 12 hours. The pumping station is located at the shore of the lake, which is about 60 ft. below the main part of the city. It is thus possible to utilize this 60-ft. head to backflush the intake with the water in the distribution system. When the stoppages became too severe it was necessary to call upon the American Steel & Wire Co.'s plant for assistance.

The conditions were investigated in 1913, by Paul Hansen, engineer of the State Water Survey, who recommended a separate intake to the inclosed water of the harbor instead of to the open lake, which intake could be built in a very short time. The location was considered favorable, there being a small bay of comparatively deep water which is inaccessible for navigation. The still water behind the breakwater would be frozen over before the season of trouble from anchor and frazil ice, and such ice would not be likely to form when the surface was frozen. Another advantage of the new intake would be its accessibility in case any obstruction should occur. The proposed new intake could be used only in emergency, owing to the polluted character of the water, but in winter (with no boats in the harbor) this condition is not much worse than at the lake intake. Mr. Hansen recommended that water from the emergency intake should never be used without purification of some kind.

The intake has since been built, for use only at such times as the lake intake is blocked, and the water is treated with hypochlorite of lime, the same as the water from the main intake.

The new intake is of 36-in. pipe, extending 593 ft. from the wet well (175 ft. from the shore) to a point about 30 ft. from the end of an old pier, where the water is about 16 ft. deep. For protection, piles were driven around the new intake.

At the shore end of the new intake pipe, just outside of the existing wet well, is a manhole with an electric-controlled hydraulically-operated valve, and inside the wet well the intake pipe is turned vertically to discharge at the bottom through a grid or screen. The valve and grid were supplied by the city. Provision is made for a future extension of the intake.

The intake opening was designed as a special elbow, covered with a screen and having an area three times as great as the pipe. The specifications, however, gave the bidder the privilege of using any other satisfactory opening, and the contractor was allowed to substitute a tubular screen, 3 ft. 3 in. diameter and 12 ft. long; this has

320 slots, ¾x12 in., and 397 round holes, ⅜ in. diameter. The bottom of this screen is about 12 in. above the lake bottom.

The emergency intake was designed by M. J. Douthitt, city engineer. Bids were invited upon both 24-in. and 36-in. cast-iron pipe. The lowest bidder was W. H. Wheeler, Beloit, Wis.: $7800 for 24-in. and $9900 for 36-in. pipe. The latter size was adopted and weighs 368 lb. per ft. The pipe was laid at an elevation of 9.8 ft. below mean lake level, from the wet well to the shore line, from which point it was laid in a trench 6 to 8 ft. deep to the outer end, where the screen is 12 in. above the lake bottom. The depth of excavation at the shore end was 15 to 16 ft. and this portion was sheeted; for about 30 ft. from the wet well the trench was drained by a pump and a series of well points.

The contractor floated the pipe in sections 96 ft. long from across the harbor, where they were poured and calked, and brought the first section up to within 30 ft. of the well. These sections were connected by means of special ball joints which a diver put in place. The work could not be started at the well as the electrically-operated valve did not arrive for two months. The contract called for the completion of the work within 90 days, but on account of a bad storm in November, 1913, and other difficulties at the shore end the work was delayed about five weeks.

The suggestion has been made that ice troubles could be overcome by building a standpipe or elevated tank capable of holding sufficient water for back-flushing the intake, but this would cost more than the emergency intake and would not provide for securing the large amount of water needed in case of large fires. This back-flushing method is employed by the water-works of the U. S. Naval Training Station at Great Lakes, Ill., but that plant has a 45,000-gal. elevated storage tank available for the purpose.

Deck-Girder Bridge with Girder Lift Span, Kamloops, B. C., over North Thompson River

A railway bridge of unusual proportions has just been finished at Kamloops, B. C., carrying the Canadian Northern Ry. over the North Thompson River. It is a deck-girder bridge, 1209 ft. long, and has a deck-girder lift span 93 ft. long. There are 12 fixed spans, also of 93-ft. length. Approaches at both ends of the bridge, of timber trestle construction, total about 1100 ft.

FIG. 1. NEW DECK GIRDER BRIDGE OF CANADIAN NORTHERN RY. OVER NORTH THOMPSON RIVER, KAMLOOPS, B. C.
(Bridge 1209 ft. long. Lift span 93 ft. long, with 55-ft. height of lift.)

FIG. 2. LIFT SPAN OF NORTH THOMPSON RIVER BRIDGE, C. N. RY.

The bridge was designed by Waddell & Harrington, of Kansas City, Mo.; the lift span is built with the arrangement and details used by them in their various lift bridges built in recent years. In this structure the lift span weighs 118 tons, and is fully counterweighted. The sixteen 2¼-in. lifting cables are equalized in the attachment to the span. Centering castings provide for keeping the span in proper alignment as it comes down to bearing, and also take the longitudinal braking thrust. The lift of the span is 53 ft., giving a 55-ft. clearance above high water. The motor is capable of raising the span in 100 seconds.

The lifting power is a gasoline engine, which, with all the machinery except the operator's levers, is located below the deck, at the middle. Limit switches coming into operation near the ends of travel of the span control the igniter circuit of the engine.

Hollow Concrete Dam for Coatesville Waterworks

New waterworks and water-supply improvement are shortly to be built for Coatesville, Penn. Designs have been made by Alexander Potter, Consulting Engineer, New York City, and bids called for. The details are still under discussion between the borough, the State Water-Supply Commission, and the State Board of Health. The work includes a hollow-concrete dam which is unusual in having its open front closed by a curtain wall; this, together with other details of the design, is planned for esthetic effect.

The dam is on Rock Run, about three miles above the present water-supply intake. It includes about 400 ft. of concrete section and 180 ft. of earth embankment, the greater part of this being on the left bank. The maximum height of the dam above natural ground surface is 35 ft. Rock is about 12 ft. below ground surface.

An 87-ft. overfall section occupies the middle of the dam. This is similar to the normal section, but with the front developed into a curved overfall of ogee form, with a short reverse curve at the bottom, and without apron.

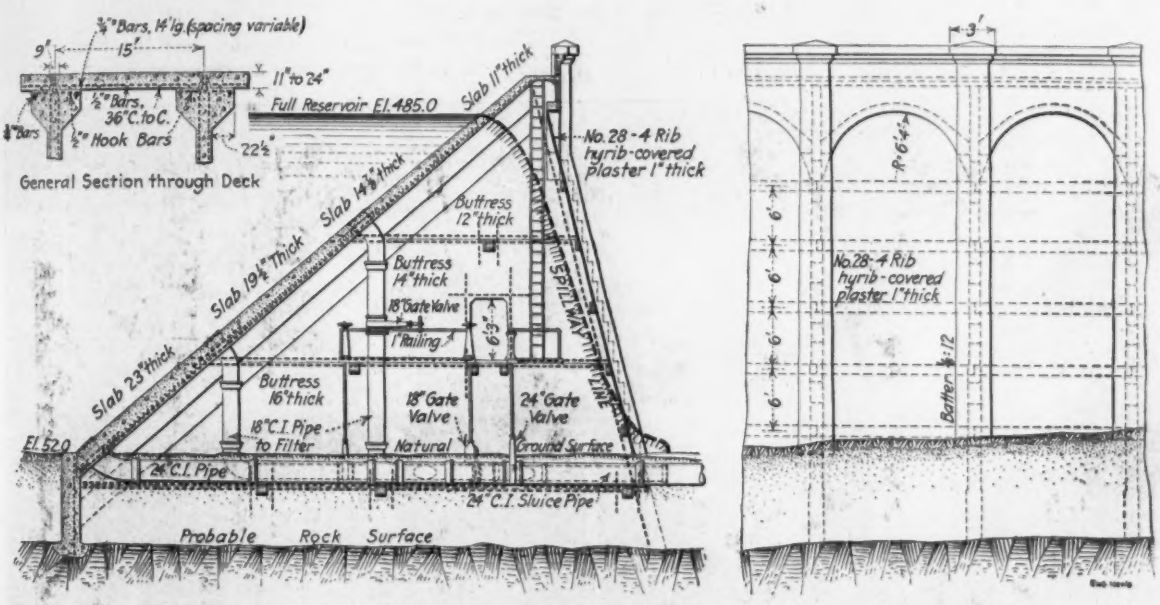
A cross-section and a part elevation of the dam are shown herewith. The buttress spacing is 15 ft.

Separately-molded reinforced-concrete beams extend across the front of the dam between buttresses, and these support a curtain wall of steel mesh and cement plaster. The upper part of the front is formed of an arched architrave extending up above the crest walkway of the dam as a parapet wall.

The walkway, parapet, and pilaster tops of the buttresses are molded separate from the main deck of the dam. The scheme of construction involves finishing the fronts of the buttresses rough and in stepped outline as shown dotted in the cross-section drawing; then forming the front portions of the buttresses, with special attention to securing good finish; forming the parapet pilasters and walkway, together with the architrave, as another operation; setting in place the separately molded girt beams for the curtain wall; and finally, placing the mesh and plastering the curtain wall.

Two constructive features of the Coatesville dam design merit special notice. First, the buttresses are carried down to rock, instead of resting on the floor as a footing. Second, the upstream cutoff wall is reinforced vertically by a sufficient amount of steel to act as a reinforced-concrete slab carrying the full hydrostatic pressure as a beam between the rock footing and the lower edge of the deck of the dam.

A special constructive detail is provided at the joint of the deck slabs over the buttresses. This is represented in our drawing by detail sketch, showing a section at right angles to deck slab and buttress. The projection above the buttress ledge is anchored down into the buttress with vertical reinforcement, and at the same time it is grooved at the side to make a thorough joint with the slabs subsequently cast against it.



HOLLOW CONCRETE DAM FOR COATESVILLE, PENN., NEW WATER-WORKS

Field and Office

Building a 115-Ft. Fire Lookout and Triangulation Tower under Difficulties

By Bristow Adams*

The Sitgreaves National Forest, Arizona, sloping gently toward the north and covered with tall timber, on the west end, affords no good natural lookout points. In the absence of such points, it becomes imperative to build towers which rise above the tree tops and allow the lookout man to look over the forest in all directions, so that the first curl of smoke from a forest fire may be detected and located, and fire fighters sent at once to extinguish it.

A triangulation station was also needed on the Chevalon District, and having in mind several points upon which the timber was only 35 to 40 ft. in height, it was planned to build where a 40- to 50-ft. tower would be sufficient. Accordingly, such tools and rigging as were at hand were thought to be adequate, and they would have been for the construction of a tower of the size we expected to build.

*United States Forest Service, Washington, D. C.

It was found, however, that there was only one point from which a satisfactory view over the forest in all directions could be obtained. Unfortunately, the timber was so tall there that it was evident that a tower must be over 100 ft. in height to be of any use.

The dangerous fire season was near at hand and we were 75 miles from any base of supplies, so it was decided to build the tower with what tools and rigging we had. We had only 300 ft. of ¾-in. rope in four pieces, the longest being 100 ft. in length; two double blocks 6 in. long and one single-sheave block of the same size.

Our tools consisted of axes, two-man saws, hatchets, crowbars, two pairs of lineman's climbers and belts, and a brace and bit. Telephone wire was used for guys.

The crew was made up of temporary employees and two rangers, under the direction of one of the forest rangers. At the start there were eight men, including one cook, one teamster, and the man in charge. By the time the tower was half built the crew was cut down to four men.

The first task was to cut and peel the timbers and skid them to the spot where the tower was to be erected.

Fig. 1. Promontory Butte Lookout Tower, Sitgreaves National Forest, Arizona

Fig. 2. Lookout Tower Showing Method of Framing

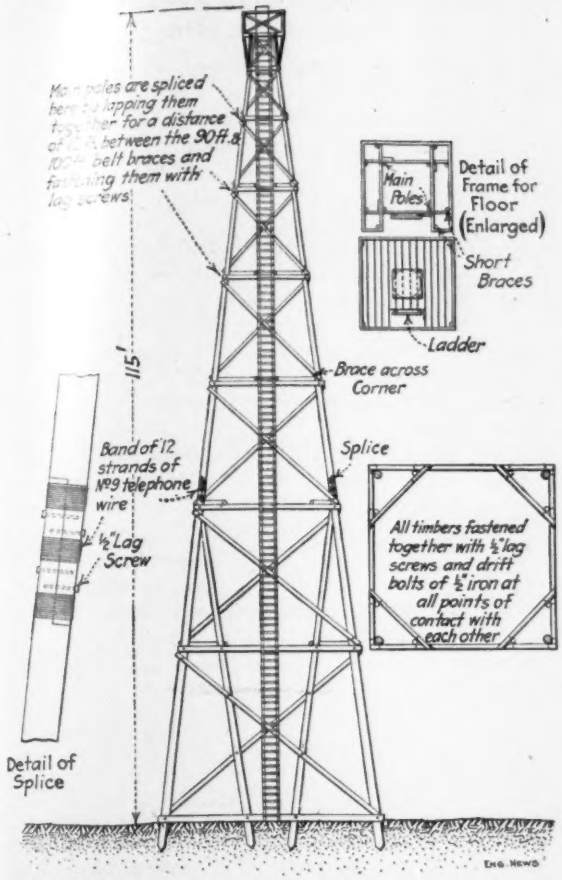

Main poles are spliced here by lapping them together for a distance of [illegible] between the 90-ft. & 100-ft. belt braces and fastening them with lag screws
Detail of Frame for Floor (Enlarged)
Main Poles
Short Braces
Ladder
115'
Brace across Corner
Band of 12 strands of No 9 telephone wire
Splice
½" Lag Screw
All timbers fastened together with ½" lag screws and drift bolts of ½" iron at all points of contact with each other
Detail of Splice
ENG. NEWS

FIG. 3. STRUCTURAL DETAILS OF THE PROMONTORY BUTTE LOOKOUT TOWER AND TRIANGULATION STATION

Much care was necessary in selecting the main poles, some of which were skidded out of dense thickets. Altogether, over 2600 lin.ft. of poles was used. The dimensions of the tower are as follows: base, 30 ft. square; platform-top, 6 ft. square; height, 115 ft.

The main corner poles averaged 16 in. in diameter at the butts and 4 in. at the tops. They are spliced at heights of 45 ft. and 100 ft. Each splice is bolted and then bound with bands of telephone wire (see Fig. 3). The timbers are fastened together with ½-in. lagscrews. The floor is constructed of 2x6-in. lumber and the rail is of 2x4-in. and 1x4-in. lumber. With these exceptions, no sawed lumber was used in the tower.

In raising the poles a tree nearly 100 ft. in height was used as a derrick mast; but since the top of this tree was not stout enough above a height of 75 to 80 ft. to carry much weight, some difficulty was experienced in placing the 55-ft. poles upon the tops of the 45-ft. ones below.

With an abundance of rigging, a derrick boom could have been rigged, which would have simplified the work to a great extent; but it must be remembered that we had only three pulley blocks, and they had to be used in hoisting each pole or brace. We were so short of rope that when we raised the second length of the main poles, it was necessary to hoist until the blocks came together, lash the pole so that it could not fall, and then stretch the tackle for another pull.

The men of the crew had no previous experience in building towers and were by no means expert climbers at first, but they improved rapidly, so that before the tower was completed, several of them were excellent men for high work.

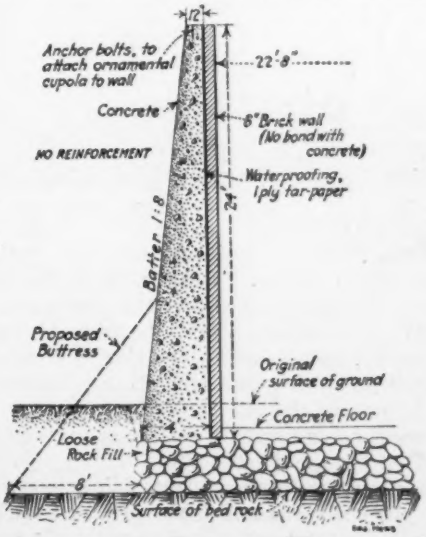
The first poles were cut on May 21, and the tower was completed and in use on June 20. The time spent in cutting and peeling logs and constructing the tower amounted to less than 24 working days of 8 hours each, for a crew which averaged five in number. In addition, a two-horse team was used 9 days.

Four galvanized-wire cables ½ in. in diameter will be added as guys. A copper cable reaching up and over the center of the tower will follow one leg to the ground and serve as a lightning arrester.

A Providentially Stable Reservoir Wall

BY LEICESTER DURHAM*

The accompanying sketch shows the details and approximate dimensions of a section through a reservoir wall which was nearly completed in this vicinity last summer. It has not yet been put into service. The reservoir is 22 ft. 8 in. square and approximately 23 ft. 4 in. deep. The wall shown was built of a rather lean concrete, of an unknown mixture of portland cement, poor



SECTION THROUGH A RESERVOIR WALL NEAR NEW PALTZ, N. Y.

sand and gravel. *There was no reinforcement.* It is proposed to build a buttress 8 ft. long at the middle of each wall and it is reported that an embankment 2 or 3 ft. high will be built around the wall on the outside.

The cost for engineering supervision of design or construction so far has been nil.

This is a private enterprise. A similarly constructed tank, of perhaps one-quarter the depth and capacity, has been in successful operation for several years. Let us hope that the new one meets with equal success.

*New Paltz, N. N. Y.

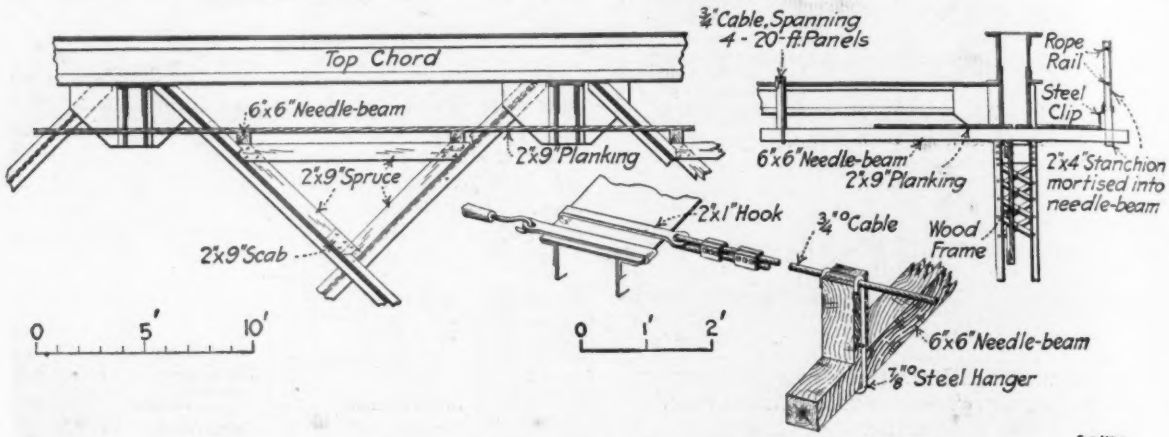
Suspension Support for Repair Staging, Williamsburg Bridge

In the strengthening of the stiffening trusses of the Williamsburg Bridge over the East River, New York City, which is now going on, an ingenious suspension scaffold is used. It was devised by John R. Simpson, foreman of riveters on the work. The drawings herewith show arrangement and typical details.

The work for which the scaffold is used is reinforcement of the top chord of the stiffening trusses by adding cover and web plates to the chord members, which are of box section. The staging is required to be alongside and about 2 ft. below the bottom of the top chord. The web system of the truss is such that support for one end of the staging is easily arranged by means of a triangular wooden frame which fits snugly between the diagonals. The horizontal timber of the frame supports two needle beams, which carry runways on both sides of the chord, being cantilevered outward about five feet. To get a support for the inner end a longitudinal cable is run parallel to the truss, about 7 ft. inside the top chord, passing over the tops of the overhead struts between the stiffening trusses and fastened to them by hook connection. Hangers from these cables support the ends of the needle beams on which the scaffold planking is laid. Wooden blocks are placed between supporting cable and needle beams to prevent the end of the beam moving upward, which might occur when the outside or cantilever end of the scaffold has the greater load.

The scaffold has proven successful in every way; it is easy to put up, move and take down, and is rigid and convenient for the work. It does not have to support very great load concentrations, as the reinforcing plates never rest on the scaffolds. These plates are raised from the roadway to the top of trusses at the center of span and from there transported on a car, specially built for this purpose, running on a wooden track laid on the top-chord cover-plate, which conveys the plate to the place where it is to be used. Such materials as boxes of rivets, riveting forges and tools, are carried on the staging, however.

The reinforcement work on the main span of the bridge is being done by department labor. Samuel Hamburger, Assistant Engineer, is resident engineer for the Department of Bridges, City of New York.



SUSPENSION SUPPORT FOR REPAIR STAGING, WILLIAMSBURG BRIDGE

Heavy Timber Pier Damaged by Ship's Collision

The kind of damage done to heavy timber pier construction by bow-on collision of a ship is shown in Figs. 1 to 4 herewith. The steamer "Matanzas," a vessel about 375 ft. long, ran into the south or down-river side of Pier 22, East River, at New York City, on Jan. 13, 1914. The day was one of the coldest of the winter. Coming up the channel east of Governor's Island 11:00 a.m the tide and wind threw the ship off her course. When she struck the pier her engines were reversed and she was moving slowly. Striking the side of the pier between the two outer transverse rows of pile supports the ship cut 36 ft. into the pier. Fig. 1 shows the gash; Fig. 2 makes it more apparent that the outer strip of the structure was forced out by the wedging action of the ship's hull.

In the standard New York Dock Department pier construction, which is used in this pier, the two outer transverse pile rows, 23 ft. apart, are double rows. In each of the double rows the piles are spaced about 2 ft. 10 in. centers on both sides of a solid diaphragm of 6x12 timbers between the piles. Transverse clamping timbers notched into the piles at the top carry longitudinals or rangers spaced 2 ft. 10 in., each consisting of two 12x12 bolted one on top of the other. On these rangers is laid 4-in. decking and 3-in. sheathing. The end pile row also has its outer side faced with 5-in. horizontal sheathing and outside of this 5-in. vertical sheathing, above low water only.

Thus the ship in cutting into the pier pushed its way 36 ft. through 7 in. of solid timber deck and in doing so cut through twenty-nine of the 12x12 ranger timbers. In pushing out the severed portion of the pier it broke twenty-one of the yellow-pine bearing piles (15 in diameter by 60 ft. long) just below the mud line, or about 35 ft. below top of pile, the water being 28 ft. deep. The fender piles, not being driven so deep into the mud, bent over without breaking; only one fender pile was broken.

The pier shed was pretty well battered in its end bay. The end is a framework of vertical 8-in. channels, 3x3 angle girts, and latticed angle struts. The first truss was 10 ft. in from the end. It was damaged by buckling in the lower chord, so that it required taking down and

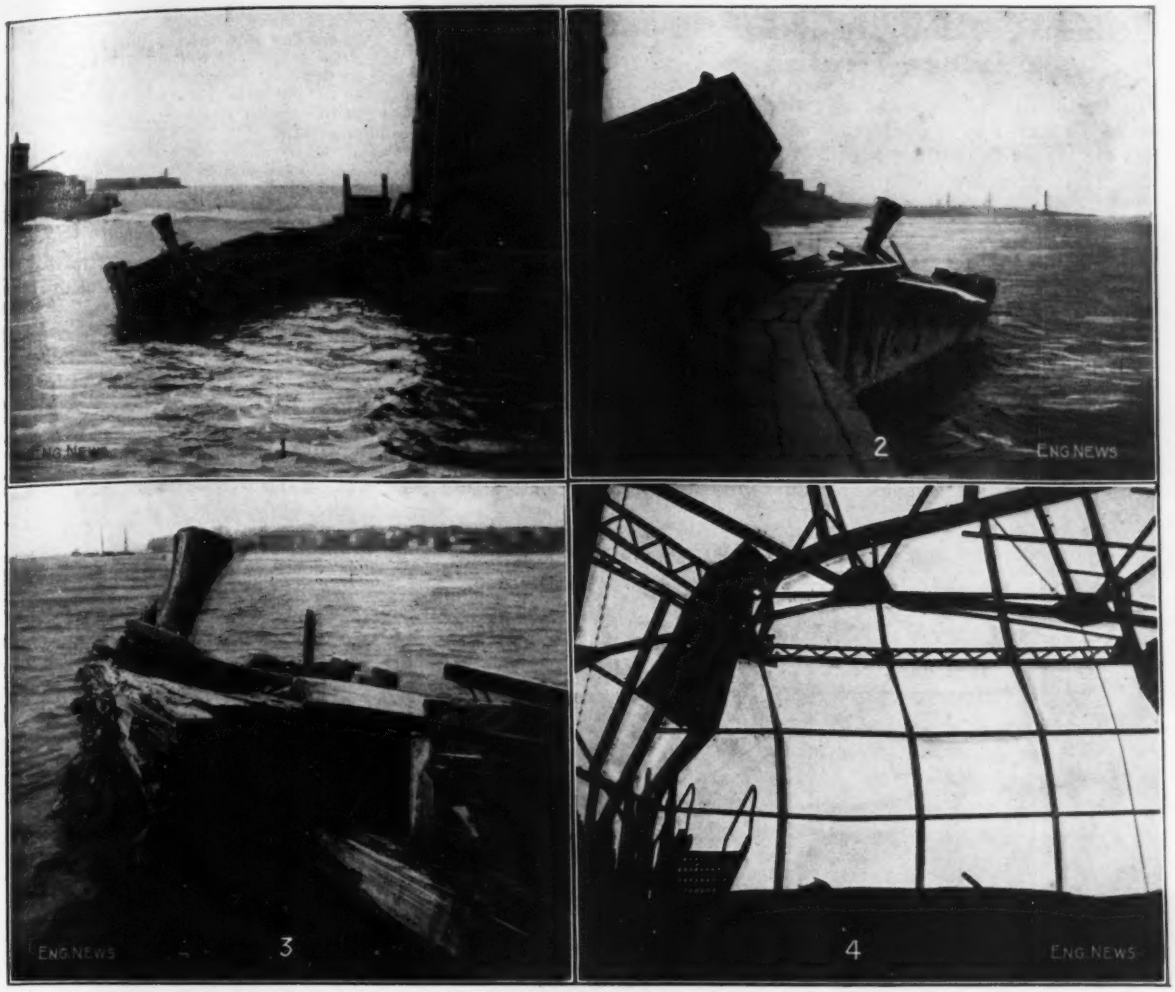


Fig. 1. Upstream view of the gash cut in the pier. Fig. 2. View along outer end of pier, showing downstream corner forced out. Fig. 3. Timber work in downstream outer corner, cut away and forced out of line. Fig. 4. View from inside toward end of pier-shed, showing damage to end framing.

DAMAGE TO HEAVY TIMBER PIER BY COLLISION OF STEAMER

straightening. The second truss from the end was only slightly buckled in the lower chord.

The ship was also damaged. Five frames were broken.

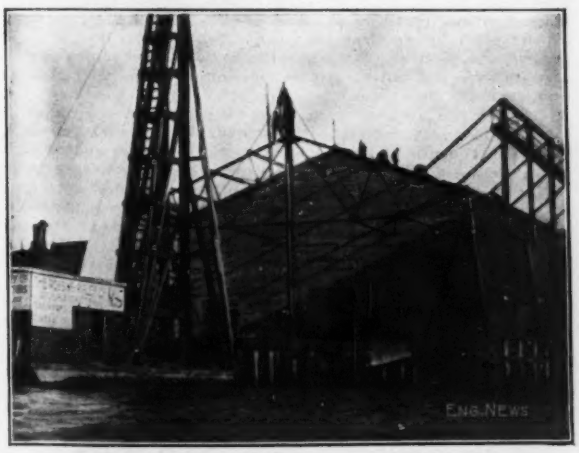


FIG. 5. REPAIR WORK ON THE DAMAGED PIER NEARLY COMPLETED

The cost of repairs to the pier is nearly $10,000. It is being done under the supervision of Chas. I. Gibbs, Jr., General Superintendent of the Ward Line. J. J. Grafflin is engineer-in-charge for the New York Dock Co., and A. L. Downey is their superintendent. The Robbins-Ripley Co. was awarded the contract for the pier reconstruction by the New York Dock Co. after Mr. Ripley had already acted as appraiser for the Ward Line.

The reconstruction of the damaged work follows the old work in every respect (Fig. 5).

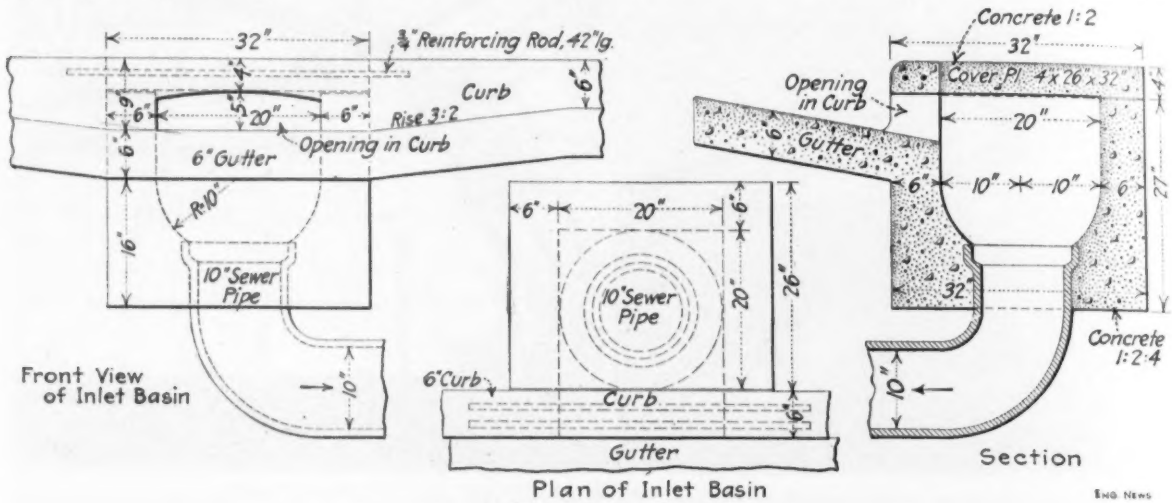
Hoisting a House-Boat Over a Bridge—The Baltimore & Ohio R.R. bridge over White River, at Washington, Ind., was washed out by the flood of March, 1913. It was replaced by a temporary trestle, as an emergency provision, to get the track across the break. Some time later the Indiana State Board of Health made a sanitary survey of the river, the party moving in a house-boat. When this boat came to the trestle it was stalled, as the bent-spacing of the trestle was 10 ft. while the house-boat was 13 ft. wide. As described by Jay A. Craven, Sanitary Engineer of the Board, before the Indiana Engineering Society recently, several schemes for getting by the bridge were considered, one of them being to pull out one of the pile bents of the trestle. It was finally decided to lift the boat over the trestle, in spite of the fact that rail level was some 35 ft. above water. A wrecking crane on the railway track handled one end of the boat and a construction crane the other end.

A Simple Curb Inlet for Street-Surface Drainage

In Carlinville, Ill., a material saving in paving work has been effected by the adoption of a simple standard inlet for taking the surface runoff of streets. Details of one of these inlets are shown in the accompanying sketch.

The curbs are concrete. The curb opening is 20 in. wide, 4 in. high at the sides and 5 in. high in the center, the top being the segment of a circle. The top of the opening is 4 in. below the top of the curb. Over the opening are two ¾-in. steel rods, 48 in. long, bedded in the concrete curb 1 in. above the opening. The sides of the concrete gutter slope gently toward the opening, so that the bottom of the gutter is depressed 3 in. across the width of the opening.

Some of the advantages claimed for this type of inlet are sanitary convenience in passing all materials, simplicity of design, low cost, assured durability due to the absence of metal parts, and the pleasing, uniform appearance given to the curb, gutter and pavement as a whole. The designer is the Morse-Warren Engineering Co., of Carlinville, where a large number have already been installed and are claimed to be giving perfect satisfaction.



A Simple Curb Inlet

Measurement of Gravel in Piles

By Oliver Clark

In purchasing, selling and excavating gravel, there seems to be a growing tendency among contractors, public officials and others to have the gravel measured in the pile as soon as convenient after it has been excavated. The immediate measurement enables the parties to know the approximate quantity so that they can arrive at an immediate settlement, and the gravel can be hauled away without the trouble of keeping account of the number of loads.

The greater number of these piles are taken from wet pits and located on comparatively level ground. The usual method of excavating is by means of an endless chain or dredge, and the manner of excavating has much to do with the shape of the pile. The more regular the pile, the more easy it is to measure it accurately.

Probably no two engineers use exactly the same method in measuring gravel piles, nor are any two piles exactly the same shape. Before attempting to measure a pile I view it carefully and determine what mathematical figure it most nearly approaches or can be made to approach by [illegible] the same into sections. Where the sides of the pile [illegible] a very steep incline, so that the taking of horizontal [illegible] vertical measurements would be difficult, I usually take [illegible] angle of inclination of the sides at the base and the dist[illegible] up the incline, from which the horizontal and vertical di[illegible]es can be computed trigonometrically.

Some piles will most nearly approach a cone, u[illegible] with an elliptical base, the area of which can be obtain[illegible] by getting the major and minor axes. The quantity can [illegible] calculated by cutting the pile into quarters, taking the [illegible] angle of inclination and the slant height from all four sides [illegible] and then computing trigonometrically the horizontal and ver[illegible]al distances. The sums of the horizontal distances obta[illegible]d from opposite sides will give each axis. The average of all four vertical heights will give the average height of the cone, at the same time taking into consideration the difference that may be in the level of the ground at all four points of the base.

If the pile should take the shape of a frustum of a cone, set a stake in the middle of the top, so that the highest point will be the vertex, and calculate as before, taking the extended volume from the result. If the pile has curved sides from top to bottom, it will be necessary to cut it into sections (parallel to the average level of the base) and calculate each section thus obtained either as a separate frustum of a cone or a prismoid, using the prismoidal formula.

The most general form of pile is oblong with steep sides curved from top to bottom, coming so nearly to an apex as to be easily extended, and having elliptical ends. My usual method in measuring such a pile is to first cut off each end by a vertical plane at right angles to the main axis of the pile, then divide the remaining middle portion into three or some multiple of three sections (depending on the regularity of the pile) by vertical planes at right angles to the main axis of the pile, so that when the area of each cross-section is obtained, the entire middle portion can be calculated as one or more prismoids.

To get the area of each cross-section, take the angle of inclination and the distance up the incline for each course in going up the pile from the base to the top (the end of each course being so taken as to correspond to the greatest break in the slope of the side of the pile), and from the data thus obtained the horizontal and vertical distances can be computed trigonometrically and the sections divided into triangles and trapezoids and the area obtained.

The end sections can be calculated as half the frustum of a cone with an elliptical base, one axis of which will be the bottom width of the end cross-section and the half of the other axis will be the horizontal distance from the end cross-section to the extreme end of the pile at the base, running in the same direction as the main axis. The method of obtaining the cubical contents of these end half cones will be the same as in the first case described.

If the pile be low and spread out over considerable ground so that the horizontal distances can be easily measured, it may be cut vertically into squares, say 10 ft. each way if very irregular or 20 ft. if regular. The elevation of each corner of the squares, as well as the elevation of the ground at the base should be taken by a leveling instrument. The

average elevation of the ground at the base should be obtained by taking elevations at equal distances around the base in such a manner as to average the high and low points as nearly as possible. The average level of the base thus obtained, taken from the average level of all four corners of each square, will give the vertical height of the prismoid whose base is that particular square. From these data the cubical contents of each prismoid and the cubical contents of the entire pile can be computed.

A Curve-Intersection Problem

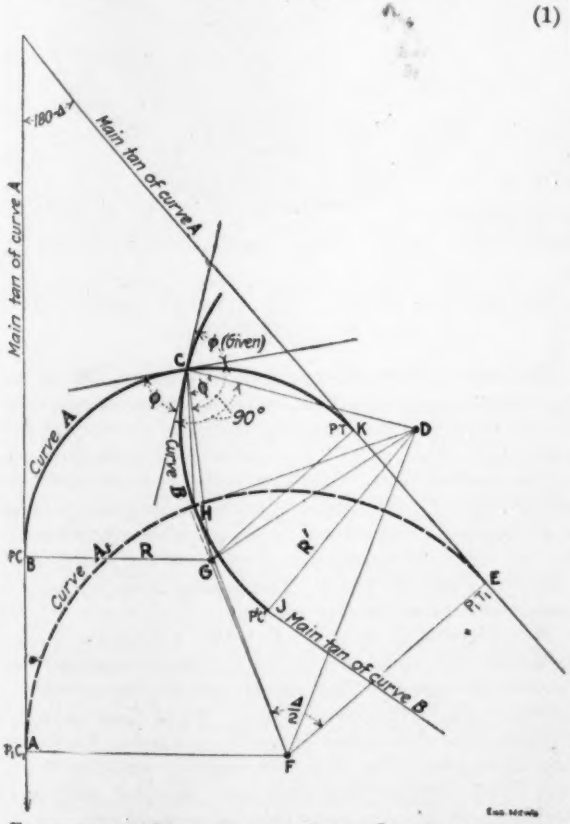
A somewhat intricate problem in curve location was given recently in a New York State civil-service examination. It may be stated thus: In two intersecting curved alignments, one curve is replaced by a curve of different degree. The new curve-intersection is to be located and its functions computed. The problem may occur in railway, canal or highway location. Graphical solution is rarely accurate enough. An analytical solution is given by B. L. G. Rees, Assistant Engineer, Public Service Commission, New York City, as follows:

In the sketch, A and B are the original curves, of radii R and C is their intersection. The revised curve is A_1 with radius R_1 and center at F. The intersection angle of the tangents of curves A and A_1 is Δ. The original curves A and B intersect at an angle ϕ. Draw triangle CGD and GFD. In triangle CGD, the sides CG $(= R)$ and CD $(= R')$ and the angle C $(= \phi)$ are known. Then

$$\tfrac{1}{2}\,(CGD + CDG) = 90^\circ - \tfrac{1}{2}\phi$$

and

$$\tan \tfrac{1}{2}\,(CGD - CDG) = \frac{R' - R}{R' + R} \tan \tfrac{1}{2}\,(90^\circ - \tfrac{1}{2}\,\phi) \qquad (1)$$



FINDING THE STATION OF A CURVE-INTERSECTION ON REVISED LOCATION

from which angles CGD and CDG are found, and DG is computed by

$$DG = R \frac{\sin \phi}{\sin CDG} \qquad (2)$$

The stationing of the original curve intersection is supposed known, so that arc BC may be found as the difference of given stations, and from the arc the angle BGC. Since

$$BGC + CGD + DGF + FGB = 360^\circ$$

the angle DGF can now be found, since BGC and CGD have been computed, and $FGB = 180 - \tfrac{1}{2}\,\Delta$. Also,

$$FG = \frac{FA - GB}{\cos \tfrac{1}{2}\,\Delta} = \frac{R_1 - R}{\cos \tfrac{1}{2}\,\Delta}$$

Then in triangle DGF the sides FG and DG and angle DGF have been computed, whence angle GFD and side FD can be computed, using equations (1) and (2). Then, in triangle HDF, the three sides are known: $HF = R_1$, $HD = R'$, and FD just computed. The angles can, therefore, be found by the formula

$$\sin \tfrac{1}{2}\,A = \sqrt{\frac{(s - b)(s - c)}{bc}}$$

which enables angles FDH and HFD to be obtained. Then, $CDH = CDG + GDF - FDH$, of which all three have been computed. Also, angle $AFH = \tfrac{1}{2}\,\Delta + GFD - HFD$. From these two angles the arc lengths CH and AH are found, and therefrom the stationing of the new intersection-point H by adding these arc lengths to the stationing of C and A respectively.

Shooting Survey Stations on the Side of a Rock Cliff—A survey party of the New York State Highway Department under the personal supervision of First Deputy Commissioner George A. Ricker recently had the job of surveying a location for a state highway across the face of Storm King Mountain, a steep bluff dipping into the Hudson River not far above the U. S. Military Academy at West Point.

This work is described in the "New York Highway News" of December, 1913, as follows:

"It has been found necessary in numerous instances to lower the men and instruments over the cliff on ropes for a distance of from 150 to 200 ft. for the purpose of taking the necessary cross-sections. For a distance of over 400 ft. it was found impossible to make cross-sections even by this method as the surrounding rock formation was inaccessible. It was, therefore, necessary to continue the line by triangulation from the tracks below.

"Measurements in some instances were made possible because of marks made in preliminary survey early in this year. The engineers could not reach certain points by rope, so they secured a field gun from the West Point Military Academy and trained this on the rocks with packages of paint as ammunition. These cans broke on the rocks and made targets upon which the State surveyors were enabled to train their instruments."

The present road over the mountain, 1340 ft. high, is perilous for automobiles and a long and difficult climb for horses. West Point and Highland Falls are practically inaccessible to residents of other parts of Orange County except by train or by traveling a long detour. From preliminary estimates as a result of a reconnaissance survey it is estimated that the projected road will cost about $100,000 per mile for four miles. Directly at the base of the mountain are the tracks of the West Shore R.R., so that it will be very difficult engineering work to cut away the rock from the face of the cliff without throwing down débris on the tracks below. It is said quarrying will be necessary for a considerable distance, for the face of the precipice is made up of huge boulders. A tunnel is out of the question, apparently, for that would destroy the chief reason for constructing the road—"a route of scenic beauty unsurpassed in this country," and of incalculable value to motor tourists.

An Instrument for Computing Azimuth from Solar Observations

A quick determination of azimuth, or stated more accurately, a determination of the sun's true bearing from either the north or south point, is made possible by the use of an instrument called the "Meridiograph," invented, patented and manufactured by Louis Ross, formerly of the U. S. Geological Survey and later U. S. Deputy Surveyor, 268 Market St., San Francisco, Calif. The instrument is shown in its elements in Fig. 1 and as it actually looks in Fig. 2. The graduations on the disks have been omitted in Fig. 1.

In form and method of operation the Meridiograph is essentially a 7-in. circular slide-rule, consisting of two graduated discs, rotating relatively to each other, a guide runner, and a specially devised transparent covering.

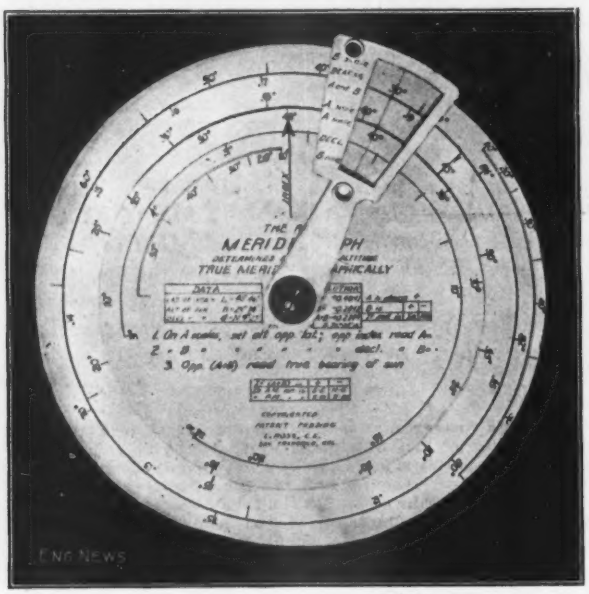


FIG. 1. THE ELEMENTS OF THE "MERIDIOGRAPH"

The disks are graduated to solve the azimuth equation. Three operations, two on the instrument and a simple addition, give the result, with a claimed accuracy of 1′.

Having given the sun's altitude measured with a surveyor's transit, the sun's declination for the time of observation (from an ephemeris), and the latitude of the point of observation, the inventor claims that anyone, without previous knowledge of astronomy, may find the true azimuth with certainty and practicable accuracy.

To use the instrument: On the *a* scales (see Fig. 1) set the latitude opposite the altitude; it is immaterial which is on which. Opposite the index on the *a* and *b* scale read the quantity *A*. Again, on the *b* scales set latitude opposite altitude. Opposite the given declination on the *a* and *b* scale read the quantity *B*. Opposite $(A + B)$ read directly the true bearing of the sun. *A* is always (+); *B* is (−) if the declination is south and (+) if the declination is north. In the actual instrument, Fig. 2, the scales are arranged in a different grouping.

The accuracy of the reading is claimed to be 1′ or 2′ of arc, depending on the time of day. The Meridiograph is good for all latitudes, and, with a small pocket ephemeris, the user may establish a true meridian, independent of books, tables, solar attachments or mathematics.

THEORY OF THE MERIDIOGRAPH

The instrument gives a mechanical solution of the formula:

$$Z' = \cos^{-1} (\tan h \cdot \tan L + \sec h \cdot \sec L \cdot \sin d)$$

which is easily derived from the formula:*

$$\cos Z = \frac{\sin d - \sin h \cdot \cos L}{\cos h \cdot \cos L}$$

Where

Z' = the sun's bearing from the south;
h = the sun's altitude;
d = the sun's declination;
L = the observer's latitude;
Z = the sun's true azimuth.

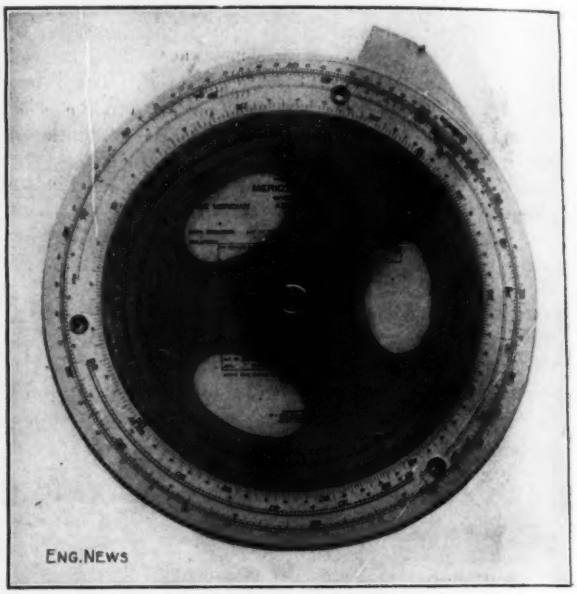


FIG. 2. THE "MERIDIOGRAPH," AN INSTRUMENT FOR SOLVING THE FORMULA FOR AZIMUTH FROM A DIRECT SOLAR OBSERVATION

The two scales *a* are logarithmic tangents, plotted on the number scale as a base, but reversed with reference to each other. Setting any two angles on these scales opposite each other gives the product *A* of their tangents on the number scale. Likewise, scales *b* are logarithmic secants, one of them reversed, while the declination scale is a sine scale. When three angles are set on these scales, the last one falls opposite the product of the secants and sine, giving the product *B*. The bearing scale is a cosine scale, which gives the final answer.

The values of *A*, *B* and $(A + B)$ will nearly always lie between 0.1 and 1.0, as given by the decimal point of the number scale. The exceptions, especially within United States latitudes, are rare. Those familiar with the slide-rule will readily grasp the method of handling these rare cases. For those not familiar with the slide-rule, brief, explicit directions are provided with the Meridiograph to cover every possible contingency.

*See article by Mr. Ross, "Azimuth by Direct Solar Observation," "Engineering News," Mar. 6, 1913, p. 454.

ACCURACY OF THE MERIODIOGRAPH—In answer to inquiries as to the accuracy of his instrument, Mr. Ross writes as follows:

Though the maximum diameter of the disks is only 7 in., the angular scales are as open, on an average, as those on an ordinary protractor of about 60 in., or 5 ft., diameter. Thus, 22½° of declination occupy 1⅛ loops, or 480° of actual arc, giving an average enlargement of 21; likewise, 70° of the bearing scale occupy about 1⅛ loops, giving an average enlargement of 8½. This effect has been attained by a suitable arrangement of logarithmic scales, and shows the fallacy of using mechanisms of watch-like fineness, with a multiplicity of axes, verniers and complex adjustments, such as are used in the solar attachment or in the "Mechanical Navigator,"* in place of applied mathematics.

The accuracy of the Meridiograph keeps pace with the accuracy of the observation, being more accurate in the early morning or late afternoon, and least accurate toward noon. To prevent the uninitiated from taking observations between 10:30 a.m. and 1:30 p.m., bearings from 0° to 20° have been omitted altogether from the scales.

The theory of construction of the Meridiograph carries no approximation. Nearly all graduations are 5′ or 10′ spaces, so that angles may be read to an accuracy of 1′ without difficulty. The mechanical centering of the disks is tested to an accuracy of about ½′. The only other possible source of error is the user's inexact reading of the numbers A, B. An allowance of 1′ to 2′ in accuracy, depending on the time of day, has been made in the description of the Meridiograph, to allow for inexact readings. The sufficiency of this allowance has been checked by actual test, on some 50 different sets of data.

TIME OR LONGITUDE—To find the change in declination since Greenwich noon, the knowledge of time need not be closer than 20 min. to 1 hr.; ordinary watch time, without any corrections whatsoever, is amply accurate.

In case the observer has not even approximate time, or in case he desires to set his watch, or determine his longitude, he may do so, to an accuracy of ½′ or better of time, by a single setting of the Meridiograph, thus:

On scales *b* set altitude against (90° — bearing); opposite declination angle read time, *t*°.

Approximate local time is (6 + *t*°/15) a.m. or (6 — *t*°/15) p.m.

All readings are made on the *b* scales only, in any order.

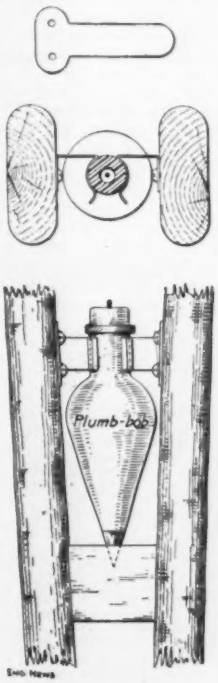
Thus: Let sun's alt. = 30°; sun's bearing = S 50°W; sun's decl. = 15°. Set 30° against 40°, both on scales *b*. Opposite 15° read 47° 40′ both on scales *b*. App. local time is 6 hr. — 3 hr. 11 min. = 2.49 p.m.

*See "Engineering News," Jan. 22, p. 180.

NOTES

Resurfacing Old Brick Pavement with Asphalt—The city of Des Moines, Iowa, has had some successful experience in renovating old brick pavements by converting them into asphalt pavements. Where the brick has a good concrete foundation only a 1-in. layer of sheet asphalt binder is added. A recent piece of work was the resurfacing of a section of Walnut St. The pavement was of brick with sand-filled joints, on an apparently secure foundation; but it had become very rough and uneven. An iron hook was used to remove the dirt from the joints and indentations, after which the surface was swept clean with brooms. A pavement surface-henter was then applied, the brick surface coated with hot liquid asphalt, and a 2-in. wearing surface of sheet asphalt applied. The work was done by the Des Moines Asphalt Paving Co., Des Moines, Iowa.

Turnouts with Spiral Curves have been laid in certain cases on the Southern Pacific Lines, the idea of the designer being to give freer movement of cars and less severe lateral thrust on the rails, thus reducing the work of maintenance. The turnouts seem to have been put in by the roadmaster without official sanction; the engineering department has no drawings of the layout and considers that there is no benefit to warrant the extra trouble and expense. The following description is from an article by C. M. Hibbetts, in the January "Maintenance of Way Bulletin," published by the Roadmasters and Maintenance of Way Association:



A BRASS SPRING PLUMB-BOB HOLDER FOR TRANSIT LEG

The tapered turnout, with a No. 10 frog, has a lead of 90 ft. 3 in., consisting of two 33-ft. rails, a 16-ft. 6-in. switch point, and a length of 7 ft. 9 in. from point of frog to toe of frog. The turnout is an 8° 30′ curve for 32 ft. from toe of frog, thence the 34-ft. taper to heel of switch point, the central angle changing 15 min. per foot of arc, that is, the first foot of rail from heel of switch point is a 15-min. curve, the second a 30-min. curve, and so forth until the 8° 30′ curve is reached.

The turnout of uniform curvature for No. 10 frog, the curve springing at heel of point and toe of frog, with 5½-in. clearance, requires a lead of 78 ft. 4½ in., and the turnout curve is 7° 40′.

At the heel of the switch point the tapered turnout is nearly straight. This greatly lessens the outward thrust of wheels, thereby reducing pressure on spikes, and while the curve is increased two-thirds of a degree in the body, this increase is in such place that the rail can be secured against spreading and overturning. When this is first introduced it is the exceptional foreman who can line it properly, as the inclination is to line straight with angle of switch point for quite a distance back of heel of point, thus giving the turnout a poor appearance and also increasing curvature. This can be remedied by giving in the book of rules a diagram similar to those for other turnouts, or by special instructions giving distances between gage side of lead and turnout rails 16 ft. and 32 ft. from the toe of frog, where taper ends. The writer tested this method on several turnouts which gave good results.

Plumb-Bob Holder for Transit Leg—The description of a device for holding a plumb-bob, in "Engineering News" of Nov. 20, has called forth a description of another, which the inventor, C. M. Lawrence, County Surveyor of Hughes County, Oklahoma, believes much superior. His directions are as follows:

Cut two pieces of thin spring brass (mine are cut from the shell of a discarded alarm clock) as shown in the accompanying sketch. Bend as shown, and fasten in the split leg with ⅜-in. round-head brass screws. I also cut from the same material an oval plate, and tack it on the block below with brass brads before boring the ⅛-in. hole for bob point. A very light grip will hold the bob, even up to a jolt which would put the instrument out of business. No parts project. And no "trigger work."

A Useful Hitch—R. C. Hardman, Colorado Springs, Colo., sends the following description of a rope hitch, which he believes has not been described previously in print:

In tying the knot a bight is taken wherever desired as shown in Fig. 1. Then catching the bight at the points a and b, the bight is turned over in a direction away from the

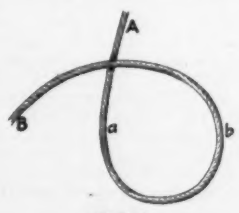


FIG. 1

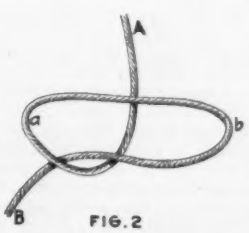


FIG. 2

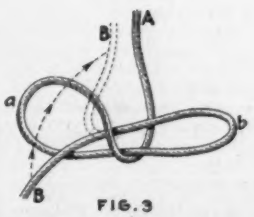
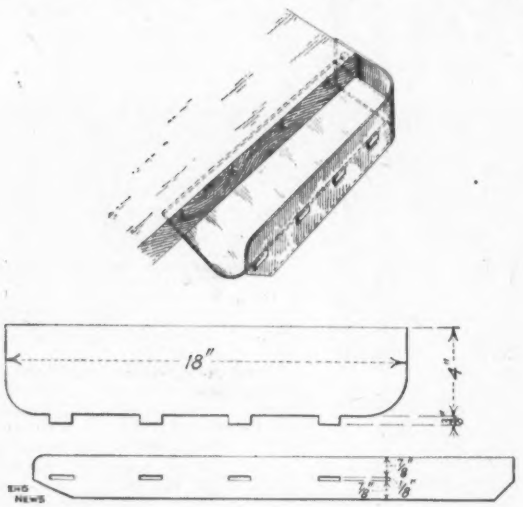


FIG. 3

FIG. 4

tier so that the knot assumes the shape shown in Fig. 2. Another turn is made in the same direction with Fig. 3 as the result. These two turns are made, in practice, in one operation, Fig. 3 being merely a 360° turn of the bight in Fig. 1. The line B is then placed beside the line A and the loops a and b are brought together back of the lines A and B. A hook or other fastening may then be placed in the double loop so made and a strain taken on the line A. The finished knot is shown in Fig. 4. Care must be taken to have the line B beside the line A or the knot will not hold. When the strain is taken off and the hook removed the knot falls apart. Its principal use is on a fall line, or any line to which power is applied, in case a temporary pull is desired at any point other than that at which power is usually applied. It may also be used in various other cases which will be obvious after the knot is known.

A Convenience for the Draftsman—Frank W. Miller, 422 Crane Ave., Detroit, Mich., has made a device illustrated herewith, which proves very useful to catch pencils, erasers, dividers, etc., which might otherwise be swept off the edge



CARDBOARD TRAY FOR HOLDING DRAWING UTENSILS, FASTENED TO DRAWING BOARD

of the board and buried under the drawings below, in case the draftsman has much shifting of layouts and reference drawings from the top of his drawing board to his reference table at the side. The device consists of two pieces of heavy cardboard, cut as shown in the sketch. The tabs left on the piece forming the bottom fit into slots in the side piece, which is held quite firmly by mashing or spreading the projecting edges of the tabs. The bottom is tacked to the under side of the drawing board along the right-hand edge, while the side piece is bent around and fastened at the upper corner as shown in the sketch. The whole thing is below the level of the board. During the last six months, Mr. Miller says, it has saved him many a search for buried instruments.

A Blueprint Finishing Machine—A machine which washes, dries and irons blueprints whether in sheets or continuous rolls, is now put on the market by the Revolute Machine Co., 417 East 93d St., New York City.* Its four principal features are a washing tank, wringer, hot drying and ironing drum, and an automatic winding device for the long prints as they come from the dryer. The machine is driven by an electric motor, through a variable-speed gear consisting of a belt running on two cones, driving a worm gear. A treadle throws the wringer out of gear. The wringer can then be reversed in direction if the print started through crooked.

The method of operation of the machine is as follows: The operator lifts a print from the tank and lays it on the upper roller (an accurate 4-in. brass tube) of the wringer. The lower roll is a felt-covered steel shaft, and bears against the brass roll. The print is fed between these rolls and the surface water pressed from the print.

A set of tripping fingers now peels the print off the brass roll and lays it on a system of traveling belts which carry it to an 8-in. steel drying drum. During three-quarters of a

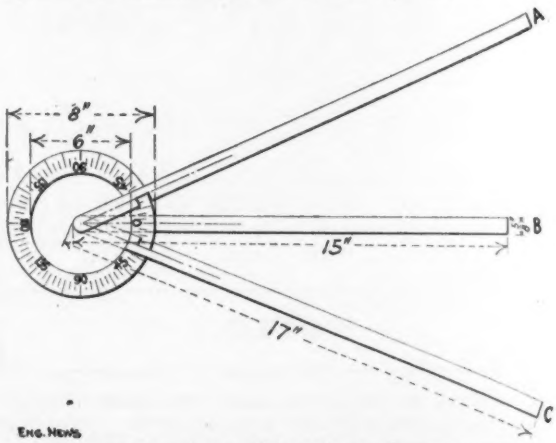
*Another blueprint finisher generally similar to the Revolute machine was described in "Eng. News," Aug. 21, p. 364. It is called the Levi Automatic Blueprint Finisher and is manufactured by the Technical Supply Co., Scranton, Penn.

THE REVOLUTE BLUEPRINT-FINISHER

revolution the paper is held against the drum. It is then either deposited in a receiving trough at the end of the machine or rolled up, depending upon its length.

This machine can be run in connection with the various Revolute printing machines, but such continuous operation is not recommended by the manufacturers. It is unlikely that the economical speeds of the two machines would be identical. The drying drum of the new machine is designed to be heated with gas, but for an additional charge provision will be made for electric or steam heating.

An Emergency Angle-Measuring Instrument of primitive construction, but sufficient for the purpose which brought it forth is described by C. V. Mann, Boulder, Colo. It was made and used by Vincent C. Colby, a private in the Philippines. A map of the territory in which the regiment was operating had to be made and there were no engineers available. Colby, an artist, was assigned to the work. The angle-measuring instrument which he built for the purpose is shown by the sketch. Circles 6 in. and 8 in. were cut out of



EMERGENCY ANGLE-MEASURING INSTRUMENT

stiff cardboard, and the outer one graduated by quartering and successive bisecting. A, B and C are three pieces of bamboo of which piece B is fastened to the bottom of the large circle, piece C is held only at the center pin, and piece A is fastened to the upper circle. The lightness of the instrument was a particular feature of advantage in climbing mountains in a country infested with Filipinos. An engineer would probably have constructed a different kind of instrument; yet this one is reported to have done satisfactory work, which is a testimonial of merit to the user.

A Small Circular Slide-Rule—A circular slide-rule 2¾ in. in diameter is being placed on the market by Lucien E. Picolet, Consulting Engineer, 19 South Ninth St., Philadelphia, Penn. The rule has direct and inverted scales identical with those on the regulation duplex slide-rules. The direct scales are used for the ordinary slide-rule computations. The universal scale and an auxiliary scale are designed to simplify the extraction of cube roots. The instrument is made of white celluloid, and sells for 50c. with direct scales only, or with both direct and inverted scales for $1.

Editorials

The absurd practice of electing the State Engineer of New York by popular vote will be stopped if a bill now apparently on its way through the New York Legislature becomes a law. The Short Ballot bill prescribing this change provides that the State Engineer and a number of other officials now chosen by popular vote shall be appointed by the Governor, without the approval of the Senate or any other body. The hundreds of engineers throughout the State would render a public and professional service by backing the bill in every legitimate way, and thus helping to stop the ridiculous practice of having a million voters pass judgment on the fitness of an engineer for office and of making an important engineering position the football of partisan politics.

In all the agitation over excessive cost, defective construction and early deterioration of the New York State highways it should not be forgotten that the work is paid for by bonds, which are to run for 50 years. Unless better work is secured in the future than in the past, the taxpayers will be called upon to pay for some of the highway work from five to ten times before the bonds are redeemed. If we mistake not the second $50,000,000 installment of New York highway bonds also runs for 50 years. Will history repeat itself in the expenditure of this money? If the law cannot be amended as regards the life of the new bonds, all the more reason why needed reforms in New York highway administration should be instituted without delay.

The introduction of a bill in the New York legislature providing for a Purchasing Department for New York City raises the question, whether a single purchasing department for a city as large as New York is desirable? It is interesting to note that in a recent report by Elihu C. Church, chief of the Bureau of Supplies and Secretary of the Department of Water-Supply, Gas and Electricity, New York City, Mr. Church expressed himself strongly as against having all the purchasing merged in a single bureau serving the whole city. Perhaps some inconsistency could be found in this argument in view of Mr. Church's strongly expressed opinion that a central bureau for his department was a great advance over the independent purchase by various divisions of the Water Department before his purchase bureau was established. Possibly the heads of the various divisions of the Water Department felt as strongly against a single purchasing bureau for the whole department as the heads of independent departments in the city government may feel at present against a single purchasing bureau for the whole city. We are, however, speaking without specific knowledge regarding the feeling on this point. The outline of the bill which was introduced at Albany to create a Purchasing Department for New York City provides that its affairs shall be administered by a board consisting of the Mayor, the Comptroller, and the President of the Board of Aldermen, and that all supplies shall be purchased subject to specifications approved by the Board of Estimate and Apportionment. The purchasing board would appoint a general purchasing agent and a general storekeeper. The plan outlined would be a radical experiment for a city of more than five million people.

Snowfalls and Street Cleaning

Recent heavy snowfalls call attention again to one of the most troublesome of administrative problems in northern cities. Under such conditions as exist in New York City, for instance, it is quite out of the question for either the city or any contractor to provide sufficient stand-by snow-removal equipment to clear any large percentage of the street area before a thaw. The inevitable result is that every heavy snowfall accompanied by cold weather is followed by snow-congested streets, and quite as certainly by a vast amount of unwarranted criticism of the city administration in general and the street-cleaning department in particular.

The critics generally overlook the fact that snow removal has to be conducted as best it may by a miscellaneous assortment of contractors' carts and wagons, filled by a still more miscellaneous aggregation of casual laborers, and that many of these men are unused to the shovel, besides being underfed, thinly clad and dispirited generally. The best that can be done under such conditions is to keep as well prepared as possible for a heavy snowfall, get the work of removal under way promptly, keep everlastingly at it—and pray for rain. The difficulties of snow removal could be greatly lightened in any city by giving the responsible head of snow removal emergency powers to carry out his difficult task free from red tape, but, unfortunately, municipal administration in the United States does not yet permit this without too much risk of adding enormously to the unavoidable high cost of snow removal. After all that can be said or done, heavy snowfalls, like heavy rains, heavy winds or extremes of temperature, must be taken as they come, and no efforts of man can prevent all the discomforts and heavy expense which they entail.

Federal Appropriations for State Road Construction

It is currently believed in Washington that the public is strongly in favor of Federal appropriations in aid of State road construction. It is probable that this belief rather than approval of the merits of the measure was responsible for the large majority by which the Shackleford Federal Aid Bill was passed by the House on Feb. 10, the vote being 284 to 42.

The widespread condemnation which the bill has received in the newspapers since its passage by the House indicates that public opinion is at least not entirely on one side. It has been said to be a foregone conclusion that some sort of a bill for Federal aid to roads will be enacted by the present Congress. It is not at all improb-

able, however, that a man whose residence is at the opposite end of Pennsylvania Ave. may have something to say concerning the legislation which will be enacted in this field.

There are, moreover, two controlling facts to be considered. The first is that any very large Federal appropriation for aid to State road building can only be made by chopping off appropriations somewhere else. The present appropriations use up practically all the revenue the government is receiving, and it is not likely that the Administration will sanction a government bond issue and an increase of the national debt in times of peace.

In the second place, if Federal aid to roads is to be of any real benefit as a popular asset to the Administration, it will not do to concentrate the Federal expenditure on a few miles of inter-city highways, chiefly for the use of automobile tourists. On the other hand, if such an appropriation for roads as the Federal treasury might possibly spare is divided up among all the States and all the roads over which United States mail carriers travel, the appropriation will be spread out so thin that very little discernible benefit will result from it.

This latter consideration is the chief criticism to be made of the Shackleford bill, apart, of course, from the criticism that the public highways are a local matter which the several States should attend to without interference by the Federal government.

Briefly summarized, the Shackleford bill provides that the Federal government shall pay to the States in aid of the construction and maintenance of macadam roads the sum of $60 per mile, for gravel and sandy-clay roads the sum of $30 per mile, and for earth roads graded and kept dragged the sum of $15 per mile, provided that at least an equal amount is appropriated by the State.

While some of the provisions of the bill are rather vague, it is apparently the intention that the work of construction or repair is to be done by the State authorities. Determination of what roads shall have the benefit of the expenditure is to be made by the Secretary of Agriculture and the State officials jointly, and the States are to furnish "such inspection and supervision of the work as the Secretary of Agriculture shall require."

It will be evident, of course, that the Shackleford bill is intended to apply to road *maintenance* rather than *construction*. The sums named above, $60, $30 and $15 a mile for macadam, gravel and earth roads, respectively, are such exceedingly small percentages of the cost of an average mile of either of these roads that if Federal aid in the above amounts were given on all the new road construction undertaken, it would represent only a small draft on the Federal treasury. Even on maintenance, $60 a mile is probably not a sixth part of the average cost per mile of maintaining water-bound macadam country highways and the same statement holds good for the $30 per mile allowed for gravel roads. It is thus only on earth roads that the $15 per mile, allowed by the Shackleford bill, represents any considerable part of the average cost of maintenance, and here, in fact, is where the great bulk of the $25,000,000 of Federal funds would be spent, if spent at all.

It is doubtful whether there is enough mileage of macadamized and gravel roads outside of incorporated cities and in use by rural free delivery carriers to enable as much as $5,000,000 of the government appropriation to be spent in maintaining them at the rates stated in the Shackleford bill. If this estimate is correct, [illegible] four-fifths of the amount appropriated by the Shackleford bill, or $20,000,000, would be distributed at the rate of $15 per mile over more than a million miles of dirt roads. Since the State has to contribute at least an equal sum, this would mean the expenditure of $30 per mile per annum in keeping the roads graded and dragged. It will be manifest that this is all such a small amount would be able to accomplish.

Desirable though this end may be, we see no reason whatever why this work should be paid for by the Federal treasury. If the local residents along a country highway do not care enough about its condition to spend $30 per mile per annum to keep its surface dragged and in proper condition for use, it is not apparent why the Federal government at Washington should step in and do the work for them.

It may be argued in favor of the bill that the Federal government would be able to bring to bear upon the work a corps of trained and experienced engineers so that the work would be done with efficiency, economy and honesty. One is staggered, however, by the problem of creating an organization to carry on, from a central office in Washington, the economical maintenance of a million and a quarter odd miles of highway spread from the Atlantic to the Pacific. It would involve an enormous expense merely to organize and equip a force to cover such a vast area. Besides, under the terms of the bill, the work is to be under control of the State authorities.

More detail criticism of the Shackleford bill is probably not worth while, in view of the probability that its passage was for political effect and that the Senate is likely to allow the bill to die a natural death, or else to amend it out of all resemblance to its original form.

Canada's Experiment in Government-Railway Construction

The report on the construction of the National Transcontinental Railway in Canada, laid before the Canadian Parliament on Feb. 12, by the Commissioners of Investigation, is a severe blow to Canadian prestige and Canadian credit.

As those of our readers familiar with Canadian railway development during the last few years are aware, a new transcontinental railway has been under construction there under government direction. The western portion, from Winnipeg to the Pacific Coast, is being built by the Grand Trunk Ry. Co. under the title of the Grand Trunk Pacific Ry. The section from Winnipeg eastward to Quebec and Moncton, New Brunswick, is being built by the Canadian government itself and is known as the National Transcontinental Ry. It is toward the construction of this eastern section that public criticism has been directed.

The report of the government commissioners appointed to investigate the construction of the railway, Messrs. F. P. Gutelius, M. Can. Soc. C. E., and J. Lynch Staunton, summarizes their conclusions as follows:

> We find that the Transcontinental Railway Commission, the Grand Trunk Pacific Ry., and those having charge of the construction did not consider it desirable or necessary to practice or encourage economy in the construction of this road.
>
> We find that, without including the money which was

unnecessarily expended in building the railway east of the St. Lawrence River, $40,000,000 at least was needlessly expended in the building of this road.

From the published abstract of the Commission's report, it appears that the losses have occurred in two principal ways: First, by building a road with a standard as to gradients, curvature and structures far more expensive than was justified by the traffic likely to be carried over the road after completion. Even if the dreams of some of its promoters were eventually realized and the line became a channel for through traffic from the grain fields of the Northwest to the Atlantic Coast, it would have been far more economical to have built a railway of moderate cost in the first place and then have reconstructed it in later years to carry heavy traffic economically when such traffic actually materialized—provided it ever did.

In the second place, the contractors on the road appear to have been favored by great liberality in the classification of material and in the allowance for overbreaking. By classing loose material as solid rock the contractors were overpaid $3,300,000. A new sub-classification of solid rock called "assembled rock," described as "fragments of rock cemented together by interstitial material," was introduced after the contract had been signed, and over $1,800,000 was paid for such material. The classification of "plowable clay" as loose rock gave contractors an excess payment of $750,000. Excavation of rock outside the section in rock cuts involved payments of over $4,000,000, and was said to have averaged 40% of the excavation.

By the use of momentum grades in location, $6,200,000 might have been saved; and $2,400,000 might have been saved by the use of somewhat sharper curvature which would yet not have impaired the usefulness or increased the cost of operation of the railway. Had wooden trestles been used in the original construction to be filled from construction trains after the line was placed in operation, some $6,000,000 might have been saved.

At different points along the line sixteen station buildings were erected, each with office accommodations sufficient for a staff to operate 500 miles of railway. It is estimated that $204,000 was needlessly expended in this way, and $300,000 in extravagant provision of freight sheds, bunk houses, store houses, etc.

The approach to the St. Lawrence River bridge at Quebec is over 1% grade on both sides of the river; but at the Chaudiere Cut, a mile east of the bridge on the south side of the river, $351,000 was spent to preserve the standard gradient of 0.4%. Illustrating the difference in cost between wooden trestles and permanent construction, at Coal Creek fill over half a million cubic yards of material were placed at a total cost with the arch of nearly $400,000. By carrying the line across this opening on a wooden trestle and postponing the construction of the arch and fill for seven years, the Investigating Commission estimates that $240,000 could have been saved.

At La Tuque, $1,000,000 might have been saved by introducing a pusher grade. The engineers of the Transcontinental Railway Commission advised it, and showed to the satisfaction of the Government, the Commission and the Grand Trunk Pacific Ry., that as it was a division point there was always at hand an engine to help a heavy train over the grade.

All parties approved this opinion, but it was deemed necessary to obtain the sanction of the government to the change. The government officials, however, advised that because the members of the government had repeatedly stated in public that the railway was being built with maximum grades of 0.4%, the economy could not be permitted.

All the above refers to the section between Winnipeg and Quebec. Of the section from Quebec eastward to Moncton, 460 miles in length, the Commission reports that there was no commercial need for its construction as it is parallel to the government-owned Intercolonial Ry. and is itself earning nothing. This section of the road, originally estimated to cost $31,250 a mile or a total of $14,375,000, actually cost $35,000,000, burdening Canada with annual interest charges of $1,200,000 a year at the present time. Had a third of this amount been expended on the improvement of the Intercolonial Ry. itself, it would have provided all the traffic facilities required by New Brunswick for many years to come.

During the original discussion of the Transcontinental Railway project in Parliament it was estimated that the total cost of the road from Winnipeg to Moncton would be $61,415,000. On Sept. 30, 1911, there had already been spent on the road $109,000,000 and Gordon Grant, the Chief Engineer, then estimated that when completed the road would cost, exclusive of interest, $151,300,000. Under the contract between the government and the Grand Trunk Pacific Ry. Co., the latter has a right to operate the road for seven years after completion without payment of rental or interest. Adding the interest charges during this period and those which have accumulated during construction of the road, the total cost to the taxpayers of Canada in 1922, when the Grand Trunk Pacific is to take it over, will have been $234,651,000. This would represent an average cost for the 1800 miles of about $130,000 per mile.

It is to be remembered in this connection that the road is a single-track line built through a wilderness where the right-of-way cost comparatively nothing. It is stated, however, that in view of the excessive cost of the road the Grand Trunk Pacific Co. may refuse to carry out its contract, involving the payment of interest charges on such an enormous sum, and the government will be left with the road on its hands and with no means of obtaining any return whatever on its vast investment.

It seems extremely doubtful whether for a long time to come the road can even earn its operating expenses. It is built through an uninhabitated wilderness located too far north for the most part, it is believed, for profitable agriculture and along a large portion of the line it is said that the timber is stunted and offers little possibilities of traffic.

Some allowance must be made, of course, in following the discussion in newspapers of this and other Canadian government matters for the intensity of partisan feeling in Canadian politics. Political considerations, unfortunately, have too often dominated in connection with railway construction in Canada.

The system of government subsidies to private railways which has always been followed in Canada is fraught with large possibilities of corruption; but the experience of the Dominion in carrying on work by the government directly has proved that there are as large or larger possibilities of graft and waste by that method.

Unfortunately, the public does not seem to learn by experience. Notwithstanding the enormous burden of

national debt piled up by the National Transcontinental railway scheme, the government is proceeding to pile it still higher by carrying out the absurd Hudson Bay enterprise which can only result in placing another load on the already overburdened taxpayers with no corresponding benefit.

It is stated that the total amount which [illegible]ada has expended in cash subsidies to private railway [illegible]mpanies and in government construction and operati[illegible] of railways and canals exceeds one and one-half bill[illegible] dollars. Most of this expenditure was incurred whe[illegible]e total population of the Dominion was less than fi[illegible]illions.

Letters to the Editor

Publicity for the Engineer

Sir—Having read Mr. Drayer's article on publicity and your editorial thereon in ENGINEERING NEWS, of Jan. 22, 1914, I am taking this occasion to tell you that I am heartily in accord with these efforts.

The engineers of Cleveland are beginning to find themselves. They are taking a more active and beneficial interest in the civic life of the community. Their advice and coöperation are being sought by committees and boards and no important public work is undertaken here but one or more of our members is consulted.

All this has been brought about by publicity, and by publicity I mean writing interesting news articles, something of general interest informing the layman in non-technical terms just what is being done and who is doing it.

Paid advertising is forbidden the engineer, architect, doctor and lawyer and is considered unprofessional. Now in my estimation, the engineer and architect have a wide and lucrative field, writing articles on new work in progress of construction.

There are engineering excursions to large manufacturing plants which would make good copy for Sunday magazine sections. If these are written clearly and authentically, they would make very interesting reading to the general public and the engineer-author would gain merited and beneficial publicity. He would then become more observant and better informed and would in time be considered an authority on these subjects. The papers would be eager for his articles and their readers would demand the same.

There is another phase of publicity that I should like to mention in this connection. Go into any bank and you will see a card beside each window informing you that Mr. Jones or Smith is in that particular compartment. This induces a more friendly relationship between customer and employee and when you find a cashier or teller who treats you courteously and with dispatch you will seek out that particular person.

Colleges are another institution which have long recognized the value of the publicity of their professors. Many of the large universities have professors who do very little teaching; they are constantly making original investigations and giving this valuable data to the world. This is publicity both for the college and the professor.

Now to return to engineering, there is a splendid opportunity at the present time for manufacturing concerns whose output is the product of a large engineering force, to give credit and publicity to the "Master Mind" in their plant, who is responsible for the success of their product. This credit will react on the manufacturing concern as in the case of the college and professor. The public will say that the well known engineer Jones designs their product and you can be sure that it will be satisfactory.

In closing, I want to quote a paragraph taken from a familiar house organ, *Dodge Idea*.

> It pays to be known. The corporation looking for a good man to add to its organization, does not go into the by-ways or the dark places, but goes into the circle of those who are known. And the fact that a man was well known a few years ago, does not stand him in good stead if in the meantime he has become a recluse. With personal prestige it is the same as with articles that are offered for sale. So long as you keep goods advertised and prominent in the public eye, they will be recognized and purchased, but discontinue this publicity and they will decline into that condition which the late Grover Cleveland once described as "innocuous desuetude." Don't cajole yourself into thinking that it is any different with a man It is not. A man must keep himself in the limelight or he will be very soon eclipsed by others who see the value of the publicity which he has ignored.

JOS. R. POE.

866 Rockefeller Bldg., Cleveland, Ohio,
Feb. 5, 1914.

A Comparison of the Salaries of Instrument Men and Clerks

Sir—There is much published in ENGINEERING NEWS concerning the salaries of engineers. Your readers may be interested in the relative rates which were paid various employees in the Government service at Panama. In the Chief Engineer's office, the engineers having responsible charge of work received from $350 to $850 per month. They in turn employed college graduates, from civil, mechanical and electrical courses of four years, at from $125 to $250 per month.

Compare with these latter figures the rates paid the clerks in the various offices, working directly under the Chief Engineer and Chief Quartermaster. The chief clerk received $350 per month. Various other clerks received $300, $200, $175, $150, $137.50, and a few $125. In the quartermaster's office rates of pay likewise ranged from $400 down to $137.50. None of these men had or needed a technical training, and few of them worked more than eight hours per day.

In comparison with the wages paid the technically trained men, note the pay received by steamshovel engineers and cranemen, who had a strong union. The operator of a steam shovel received from $210 to $240 per month. The cranemen $175 to $190.

A rodman received $83.33 per month. After six months, if he survived the bush, malaria, exposure to

rain, or beans and rice, he might get $100. Most of these men put in ten and twelve hours work per day, and lived in camp. Some were engaged in the lock and spillway construction and had better quarters, but few if any, received over $150 per month.

If the Government undertakes the construction of the Alaska railroad, it is to be hoped that the engineers in charge will see that technically trained men, engaged in either surveying, or exploration under such severe conditions, or those having the design of structures, and supervision of construction, will receive at least as much as a bunch of clerks performing timekeeping or another bunch whose principal responsibilities are to take telephone messages, for eight hours per day.

E. A. V., Jr.,
Jun., Am. Soc. C. E.

New Orleans, La., Jan. 26, 1914.

The Construction of the Hampden R.R.

Sir—Your editorial of Jan. 29, entitled "The Financial Responsibility of Directors," is timely and points a very good moral; but some of your statements regarding the Hampden R.R. appear to be based on assumptions not exactly in accordance with facts.

Referring to this road, which was constructed in order to give the Boston & Maine R.R. an entrance into Springfield and a direct route to New York, you make the following statement:

> **The Athol branch of the Boston & Albany R.R. parallels the Hampden R.R. for nearly its whole length. As the New Haven held a 50% controlling interest in the Boston & Albany, it was in a position to at least make friendly negotiations for the use of this stretch of single track for the few through-passenger trains that it might have desired to operate in this way. Such an economical solution of the problem appears not to have been even thought of.**

As a matter of fact the possibility of such a solution was investigated quite thoroughly, and surveys were made for a possible physical connection of the B. & M. with the Athol. The Athol branch is, however, a very inferior piece of track, pursuing a sinuous course through a narrow river valley, with numerous sharp curves and some heavy grades. It certainly could not be used for fast passenger service without very costly improvements.

Regarding the high cost of construction of the Hampden R.R. you say:

> **It may be thought that the high cost of the road might be due to its being a double-track line of phenomenally heavy construction. It appears, however, that the line is only single track.**

Although there is only a single track laid at present, most of the roadbed was graded for double track, and there was a great deal of very heavy construction, many of the cuts being 50 ft. or more in depth for long distances, and the fills of equal volume. There are two long steel viaducts, one of them, crossing the Chicopee River, being about half a mile over all, with a single span over the river of 300 ft.

The Hampden road has a ruling grade of 1.23%, yet there are only six curves in the entire fifteen miles, the sharpest curve being but 4%. There is one stretch of tangent over five miles in length.

While the cost of upwards of $260,000 per mile is large, and probably represents a considerable profit to the promoters, it is not so excessive as might be inferred from the statements of some persons who have never seen the road and cannot appreciate the very heavy construction work which was done on it.

K. P. Armstrong.

7 Water St., Boston, Mass., Feb. 5, 1914.

[There is no doubt that the Hampden R.R. represents very heavy construction work and was built with very high standards as to gradients and curvature; but no satisfactory demonstration has been made that this large expenditure was justified.

It is true the Athol Branch has much curvature and a broken grade; but we are informed that its curves and grades are after all little, if any worse, than those on the Central Massachusetts Ry. with which the Hampden R.R. connects. There was here, moreover, no question of the economy in operation of heavy through freight trains which has been the governing factor in most work where heavy expense has been undertaken in railway reconstruction. The main purpose of the Hampden R.R. was to facilitate the movement of through passenger trains between New York and New England mountain and coast resorts during the summer season. The expenditure of a small part of the cost of the Hampden R.R. in improving the Athol Branch would have certainly made it as good as or better than the Central Massachusetts. Even if it was deemed necessary to build an entirely new line, the testimony of Edward C. Sherman, the engineer who investigated the matter for the Massachusetts Public Service Commission, showed that the road could have been built on a different location with excellent grades and alignment for about a million and a half dollars less than the Hampden R.R. cost. Further, on the Hampden R.R., itself, by making two detours and some slight changes in grade, half a million could have been saved.

It is at least interesting to know in connection with this very heavy work on the Hampden R.R., that the Woronoco Construction Co. which had the contract for the construction sublet the rock excavation at a profit of 40 to 50c. per cu.yd., and the earthwork at 12 to 15c.—Ed.]

Scope of the Prismoid Formula Mathematically Defined

Sir—In your issue of Dec. 11 was an inquiry by W. J. H., of Newburgh, N. J., regarding the scope of the prismoid formula. This formula seems to be more or less of a joker inasmuch as it holds true for solids to which the definition of the prismoid does not properly apply.

The definition usually given is as follows: "The prismoid, or prismatoid, is a solid, bounded by plane rectilinear figures, and has for its bases two polygons of the same or of a different number of sides which lie in parallel planes." Allen, in his "Railroad Curves and Earthwork," gives essentially this definition and goes on to show that the formula applies to a section of a complex solid with a curved surface, called the hyperbolic paraboloid. He also makes the conventional statement that it applies to many solids having curved surfaces. The question naturally arises, "Where does it all end"? The following is intended to indicate definitely the scope of the prismoid formula.

Suppose the solid under investigation, is placed in a system of rectangular coördinates, with its base in the

XZ plane, and suppose it is possible to express the area of any section parallel to and distant "y" from the base as:

$$f(y) = A + By + Cy^2 + Dy^3 + \ldots . Ry^n$$

The volume of such a solid would be

$$V = \int_o^h f(y)\, dy$$

$$= h\left(A + \frac{Bh}{2} + \frac{Ch^2}{3} + \frac{Dh^3}{4} + \frac{Eh^4}{5} + \ldots\ldots \frac{Ry^n}{n+1}\right)$$

Suppose we now apply the prismoid formula to the same solid. The area of the lower base would be $f(0)$, that of the upper base $f(h)$, and that of the midsection $f\left(\frac{h}{2}\right)$. Accordingly, if the formula holds true for such a solid, its volume, after simplification, must be

$$V = \frac{h}{6}\left(f(0) + 4f\left(\frac{h}{2}\right) + f(h)\right) =$$

$$h\left(A + \frac{Bh}{2} + \frac{Ch^2}{3} + \frac{Dh^3}{4} + \frac{5\,Eh^4}{24} + \Sigma Fh^5 + \ldots . \Sigma Rh^n\right)$$

These two expressions for the volume are identical up to and including the term in "h" of the third degree. This means that, if the prismoid formula is to give the volume of the solid, it is a necessary and sufficient condition that we be able to express the area of any section parallel to and distant "y" from the base, in a rational integral, algebraic function of no higher than the third degree in "y". Remove the assumption made at the beginning and a little thought will show that the above condition is a sufficient although not a necessary one.

Consider now the number of solids to which the formula applies. The area of any section parallel to the base, according to the above demonstration is

$$f(y) = A + By + Cy^2 + Dy^3$$

It is, of course, possible that some of the coefficients be zero, in which case the area of the section might be $f(y) = A$, or $f(y) = A + Cy^2$, or $f(y) = By + Cy^2 + Dy^3$

In short, there will be as many varieties as there are possible combinations of four things, taken four, three, two or one at a time; i.e., $2^4 - 1 = 15$. The condition when all the coefficients are zero, is excluded. Corresponding to each of these there will be *at least one* solid with volume given by the prismoid formula.

Again, given any curve $y = f(x)$, if $\pi[f(x)]^2$ is a rational, integral, algebraic function of no higher than the third degree in x, then the prismoid formula applies to the solid of revolution generated by that curve. Therefore, the volume of any solid, or any of its frustra between parallel planes, generated by any conic may be found by means of this formula. This explains the fact that the volume of the sphere, ellipsoid, paraboloid, etc., may be found by the prismoid formula.

These considerations make it possible to present the prismoid formula in another form. Given that the area of any section parallel to the base of a solid is

$$f(y) = A + By + Cy^2 + Dy^3$$

where A, B, C, and D are known; volume of the solid is

$$V = h\left(A + \frac{Bh}{2} + \frac{Ch^2}{3} + \frac{Dh^3}{4}\right)$$

HARRY ROESER.

Oklahoma A. & M. College, Stillwater, Okla.

Dec. 12, 1913

Concerning Bridge Engineering Competition

Sir—The writer has just finished reading your article entitled, "Another Bridge Engineering Competition," in your issue of Feb. 5, and cannot refrain from taking exceptions to some insinuations you incorporate in the article.

The gist of your objection seems to be that the committee handling that work has not furnished enough data for an engineer to prepare a design for the bridge. You also express the great fear that some engineer engaged in bridge building might secure the designing of this work, or in other words the whole article tends to create the impression that no engineer engaged in bridge building is capable enough or honest enough to design bridges.

The writer might state here that neither his company nor himself has any interest in the design for this work, nor do they intend to have, but he cannot help but express himself at your distinction between engineers and contractors, and since the matter has been brought up, to ask you a few questions regarding this distinction.

You say in the article in question that the "Committee expects the bridge building companies, who are used to taking gambling risks of this sort, will come forward on Feb. 18, with various ornamental designs for the proposed bridge, from which the committee will proceed to make its selection, aided by the expert advice of the respective bridge salesmen."

You also say, "the direct tendency is to throw work into the hands of concerns with the lowest professional standards or none at all, who often combine the functions of the engineer and the contractor. Men of greater ability and higher standards of honesty are driven into some other occupation; and such an outcome is nothing short of a public calamity."

You will greatly favor the writer by answering a few of the following questions:

Why is it impossible for a contractor to be a capable engineer and *vice versa*?

Why is it impossible for a contractor to do a legitimate and honest business?

Why is it impossible for a bridge contractor to be a capable engineer and a man with high standards?

Why is a man who calls himself a Consulting Engineer, more capable than the man who leaves off the title "Consulting"?

How can a consulting engineer tell what work will cost when his whole knowledge of costs is based on a handbook some years out of date and dealing with particular cases?

Being a contracting bridge engineer, the writer often hears the insinuations that all contractors are crooks and that if a contractor is making any money he should be investigated; but you can imagine his surprise when reading practically the same things in your magazine.

You say that dishonesty is prevalent among bridge contractors. Would not the word "found" be a little closer to the truth?

And while on the subject the writer would like to ask a few more questions as to what you call "honesty."

Is it honest for a consulting engineer to design a structure, make an estimate far too low for good construction, and far below what a contractor can build the work for, and then tell the owners that the contractor is asking an exorbitant price?

Is it honest for an engineer to design work, inspect all of the material that goes into it during the construction, and then require the contractor to guarantee the structure both as to design and construction? Is it honest for the engineer to place all of the blame on the contractor when anything goes wrong?

Is it honest for an engineer to take the owner's money for designing a structure with which he is not familiar, as for example a drainage engineer will undertake the design of a bridge when his knowledge of bridge designs consists of what he got from his textbooks when he was in school some years before?

Why does an engineer get paid for taking soundings, etc., and then require the contractor to do this work all over again; the engineer giving absolutely no guarantee and taking absolutely no responsibility?

In this case of the Chattanooga bridge, why is it harder on the engineer to make his own soundings than it is on the contractor? Suppose this committee went ahead and hired an engineer on his reputation as you suggest. He will prepare a design including plans and specifications. On these he will show the results of his investigations and soundings, and will then immediately say that these facts are approximate only and that the contractor must satisfy himself as to the nature of the foundations, etc., and that the engineer can be responsible for nothing.

If the contractor goes ahead and makes these soundings he very justly adds the cost of them to his bid. Thus the owners are paying for the same work twice. Why doesn't the engineer assume some small responsibility himself since he gets money for doing something? Perhaps it is this lack of responsibility that helps him keep "honest."

Why is it that engineers almost invariably place in their specifications the clause making them the final arbitor in case of dispute, instead of putting it up to some disinterested party? Is it because he considers his own judgment superior to that of all other engineers? Is it a sense of honesty that places the contractor absolutely in the power of the engineer wherever the engineer finds it possible to secure this power?

If you are going into this matter and finding fault with competition that requires engineers to make their own investigations, why not take up the matter of requiring contractors to make costly investigations of sites that have already been tested by the engineer who has been paid to do this work.

The writer will admit that there are some dishonest contractors, such as you will find in every line of business, but why do you say that such men are "prevalent"? There are evils in the business that should be done away with but these evil practices are as much at home with the consulting engineer as with the contractor. Furthermore, it is the writer's opinion that engineers of high principles and capabilities will be found both among the engineers and the engineering contractors or contracting engineers. And a close investigation will probably show that the engineers who are also contractors are responsible for by far the greater amount of progress which is made in the matter of economical and satisfactory construction.

The writer has been speaking from an experience that covers practically the whole of our country and has not gaged his view point from a few local conditions.

If you will kindly throw a little light on some of the questions the writer has asked you it will be greatly appreciated.

WM. G. MORRISON.

Des Moines, Iowa, Feb. 7, 1914.

[Our correspondent appears to have read into our editorial ideas entirely foreign to its purpose and intent. We have at no time said, or insinuated, that it was impossible for a bridge contractor or a bridge builder to be at the same time a capable engineer with high standards of honor and honesty. Any such statement would be absurd on its face in view of the many engineers of high standing connected with bridge-building companies in responsible positions.

We did say, however, and strongly maintain, that both the bridge engineer and the contracting engineer are needed. Further, we believe the best bridge-building concerns would much prefer to undertake bridge work upon the plans and specifications prepared by a thoroughly competent engineer than engage in the competition for a bridge contract let by some political body, wholly unaided by any engineering advice.

We make no defense of incompetent engineers or of those who undertake work as consulting engineers with inadequate ability and experience; but we doubt whether even our correspondent would care to maintain the proposition that all bridge engineers not engaged in contracting work are incompetent, or even that all bridge work should be let to contractors without the intervention of an engineer.

As for the abuses mentioned by our correspondent, consisting in placing upon the contractor responsibility not properly belonging to him, etc., these have been often pointed out and condemned in these columns.

These questions, however, are apart from the main issue, which is the method of competition adopted by the Chattanooga bridge committee. Laying aside for a moment the whole question of contractors vs. engineers, our correspondent says that the gist of our objection was that the committee had not obtained surveys and soundings at the bridge site, and made them available to all competitors proposing to submit plans and estimates. Taking even our correspondent's position, that the competition is to be between bridge contractors and not bridge engineers, it would seem to us in the interest of these also that the committee should have had the surveys made once for all, instead of a dozen competitors each spending the money to have the work done. Our editorial was prepared with the interest in mind of the honest and reputable bridge contractor (who we believe is typical and not exceptional) as well as of the engineer who earns his living by professional work; and it is a matter of surprise to us that our correspondent should have interpreted it to the contrary.—ED.]

Sir—Regarding your recent article on the competition for the Chattanooga Bridge, I am quite sure that engineers throughout the country are in accord with you for the improvement and better regulation of competitive work among engineers, but I am also sure that it will be a hopeless task unless some official action is taken by our National societies.

R. A. C.

Pittsburgh, Penn., Feb. 12, 1913.

An Emergency Snow Plow for Opening Street Gutters

During a recent heavy snow in Detroit, Mich., a 6-ton garbage-collection motor truck was equipped with a snow plow and used to open up snow-blocked street gutters, as shown by the accompanying view. For balancing to buck against snowdrifts the truck was partly loaded with gar-

AN EMERGENCY PLOW FOR STREET USE, DETROIT, MICH.

bage boxes. We are informed by George H. Fenkell, Commissioner of Public Works, of Detroit, that the snow plow was rigged up by John J. Knight, superintendent of Street Cleaning and Sanitation, and that "it does excellent work."

A Bill to Promote the Development of Water Powers on the Public Lands

A bill (Senate No. 4415) has been introduced into the U. S. Senate by Senator W. L. Jones, of Washington, with the object of encouraging the development of water powers on the public lands. The important provisions of the bill are briefly as follows.

Rights are granted upon the public lands to "any state or municipal subdivision thereof or to any public-service corporation" to "construct, develop, maintain, and operate water-storage reservoirs, canals, conduits, water-power or hydro-electric plants, transmission and distributing lines," etc. These rights are secured by filing approved plans and specifications with the Secretary of the department having jurisdiction.

To avoid the problem of whether or not the Government can levy a tax on power produced, this bill imposes an annual rental charge of 5% on the market value of the lands before occupation. Within 90 days after the approval of the plans and specifications, the grantee and the Secretary must agree on the fair market value or else the grantee may begin proceedings in the U. S. District Court for the locality, to determine this value. These courts are given jurisdiction by the bill to proceed according to laws and rules in force for the exercise of eminent domain.

The rights thus secured are to continue for 50 years, or until compensation has been made to the grantee for the fair value of the property, not including anything for the rights in question. At the expiration of the 50-year period, Congress may terminate the rights involved and provide for taking over all of the property of the grantee by the United States, by any State within which the land is located or by any municipal subdivision. But the fair value of the property has to be paid [illegible] all contracts approved by public authorities having [illegible]sdiction have to be assumed. This fair value must [illegible]y agreement with the Secretary of the department [illegible]g jurisdiction or by proceedings in a U. S. District [illegible]t.

In order to prevent tying up water-rights [illegible]speculators, a grantee must begin construction with[illegible]o years of the date of agreement about, or final adju[illegible]tion of, the land value. Construction must be comple[illegible] in five years sufficiently to supply the reasonable n[illegible] of the market. However, extensions of time may be [illegible]nted by the President in the public interest.

It is provided that the rates shall be reas[illegible]ble and the service adequate. Courts and public-ser[illegible]e commissions are directed to include the 5% rental [illegible] land in the basis of rate making. The bill reserves to the United States the right of rate and service supervision, if the business becomes interstate or if state and local authorities fail to regulate.

All water-power reserves would be opened up by the President under the appropriate public-land laws, so far as this action would not impair or destroy use for power development, transmission or utilization, water power being made the dominant use and all transfers of land being subject to such reservation.

The bill enjoins all ownership or control of works to form an unlawful combination or contract to limit the output of energy.

Methods of Electrolysis Mitigation

Public-utility engineers generally, but more particularly gas and water-works men, still face the serious deterioration of their buried pipes by stray currents from electric railways. To all such men, a bulletin (No. 27, "Electrolysis Mitigation in Springfield, Ohio) recently issued by the U. S. Bureau of Standards will be interesting because it presents the conclusions of the physicists of the Bureau on the several available means of mitigating this evil. The Bureau has better resources for making a comprehensive and thorough study of these widespread problems than any other organization and evidently has made the most of its opportunities. Its determinations, therefore, are of unusual weight in mitigation work, planned or under way.

The bulletin, ostensibly, is a study of the conditions in Springfield, Ohio, where a typical problem was found and an attempt to relieve conditions by the use of insulated return feeders between various points of the track and the power house (the scheme favored by the Bureau). Only some 18 of the 55 pages in the bulletin, however, treat of the Springfield work, the rest describing and commenting on the several mitigation schemes which are seen about the country. Indeed, this general discussion is the more important part to most readers. The following extracts have been made to show in as concise form as possible the attitude of the Bureau investigators and their reasons therefor. (In a few instances the language has been slightly changed to secure continuity of excerpts and to increase clearness.)

METHODS APPLICABLE TO PIPES

SURFACE INSULATION OF PIPES—Painting or otherwise insulating the surface of pipes was early resorted to and is still used in some instances. About 40 different kinds

of paints have been tested and of these not a single one has withstood the action of the very moderate test potential of four volts for any considerable length of time, failure of the coating with consequent pitting of the pipe occurring within a few months in all cases. None of the paints tested are absolutely impervious to moisture. The coating becomes slightly conducting and a trace of current flows, at first giving rise to slight electrolysis and more or less gas beneath the coating. As this gas increases, the coating is ruptured, the current is increased and rapid electrolysis of the exposed iron follows. If the coating is sufficiently porous, it may remain intact and electrolysis may continue beneath.

Coating pipes with treated textiles or tarred paper is open to the same objections though the time required for initial failure is usually greater. We have tested a considerable number of such coatings with uniformly disappointing results.

INSULATING JOINTS IN PIPES—This method has found extensive application in recent years. It is but natural that the initial attempts should have resulted in some disappointments. If properly installed, with joints of proper construction and used with sufficient frequency, it can be made very effective in minimizing electrolysis troubles. A third or a fourth of the joints insulated is usually sufficient, even under severe voltage conditions, while often a much smaller percentage would suffice. In the case of old systems the expense becomes great, unless the potential gradients in the earth are first reduced by other means so that a comparatively small number of insulating joints will be sufficient.

PIPE-DRAINAGE SCHEMES—A system of wider application in this country than any other is "pipe drainage." This method has taken a variety of forms.

(1) Direct Taps Between Pipes and Rails—At various points throughout the positive areas where the pipes are close to the tracks, they are connected by short, heavy cables. The potential gradients along the pipes throughout the region affected are the same as in the rails. Commonly this produces heavy currents in the pipes and, in case occasional high-resistance joints are encountered, the large leakage of current around the joint will cause rapid injury to the pipes.

(2) Negative Feeders to Pipes—Another scheme consists in running negative feeders direct to various points of the pipe system. In some cases it has been proposed to run these long feeders parallel to the pipes and tap them to the pipes at frequent intervals. In some installations boosters have been connected to the longer feeders, in which case they can be made much smaller and any desired amount of current can be taken from the pipes by each feeder.

Any form of pipe drainage will necessarily increase to a greater or less degree the amount of current carried by the pipes, which is accompanied by the ever-present danger of trouble developing on high-resistance joints at obscure and unlooked-for places. Connection of the pipe system to the busbars or rails lowers the potential of the pipes and tends to make them negative to other pipe and lead-cable systems, thereby endangering the latter. Sometimes it is practicable to tie in all pipe and table systems and sometimes not; in any case it would greatly extend the area in which acute troubles might be expected. This method increases the life and fire hazard, particularly in connection with gas mains.

The method requires that all pipe systems to which it is applied shall be electrically continuous throughout; but in a great many installations, cement, rubber-gasket and other insulating joints are now largely used.

There is a tendency toward the production of an excessive amount of alkali at the surface of a negative electrode imbedded in earth. Concentration of alkali will have no detrimental effect on iron pipe, but may tend considerably to increase the self-corrosion of lead service pipe.

The greatest objection to the pipe-drainage system, and to all the other methods mentioned above, is that they are designed to relieve the symptoms rather than to remove the cause. As a rule, in the early stages of its application, the effect is apt to be apparently beneficial, reducing the danger in positive areas more than it increases it elsewhere. As the railway grows, the current in the pipes may become so great that the consequent damage will be of greater moment than the reduction of troubles in the positive areas.

Among other methods proposed may be mentioned (a) surrounding the pipes with certain inhibitive chemicals, such as lime, etc., (b) use of cement coatings on the pipes, (c) maintaining pipes negative to certain structures by means of battery, motor generator, etc., (d) favorable location of pipes with respect to rails, (e) use of noncorrodable conducting coatings, and (f) electric screens—sheets of metal placed near to or surrounding a portion of the pipe and electrically connected thereto. Our investigations have convinced us that none of these methods can be considered seriously.

METHODS APPLICABLE TO THE RAILWAY SYSTEM

In the systems mentioned above, currents are permitted to stray from the tracks without restriction. The sole aim is either to prevent their entrance into the pipe or to provide means for their exit with as little injury to the pipes as is possible. It would appear more logical to prevent, to a large extent, the leakage of the currents from the railway return conductors into the earth.

There is no question but that the double-trolley system, if properly installed, would entirely eliminate electrolysis from railway currents, but the cost of installation does not appear to be justified.

TRACK AND ROADBED—In recent years, engineers have come to recognize the futility of trying to maintain a proper state of track conductivity by merely bridging the joints with short copper bonds. The tendency now is to make the joint itself electrically continuous rather than to shunt around it. Cross bonding between the rails is also much resorted to. All special work should be shunted by copper cables capable of carrying all of the current passing over the tracks at that point. In some cases the drop across special work has given rise to very serious electrolysis.

A properly drained roadbed is also a very effective aid in reducing the leakage of stray currents from the rails.

UNINSULATED-NEGATIVE-FEEDER SCHEME—This method has been much used to increase the conductance of the track. With feeders costing $4000 per mile for cables of 1,000,000-circ. mil. section installed, the most economical potential gradient to allow in the rails is about three volts effective per 1000 ft. (root-mean-square for 18-hr. period) with power costing 1c. per kw.-hr. Wherever the potential gradient exceeds that figure, the addition of copper in parallel with the rails produces an annual saving more than sufficient to pay all proper charges, including interest, taxes and depreciation, on the copper required to reduce the potential gradient to three volts. When this value is exceeded in any locality, that section of the track is not only being operated at an unnecessary loss, but electrolysis conditions are unnecessarily bad. In carrying the voltage limit below this value, the cost of the necessary copper soon becomes prohibitive. On the other hand, a potential gradient of three volts per 1000 ft. is much too high to permit a reasonable degree of immunity from electrolysis.

INSULATED NEGATIVE-FEEDERS—In this scheme the connection at the power house is either removed or given a suitable high resistance, and insulated feeders are run from the negative bus to various points of the track. Current being taken off of the rails at numerous points, high current densities and high potential gradients in the rail are avoided. The actual drop of potential on the different feeders is of secondary importance so long as it is practically the same in all; then any desired potential restrictions can be imposed on the track with freedom to design the feeders to give maximum economy, which cannot be done when the feeders are connected in parallel with the tracks. If, instead of making the drop on all of the feeders the same, we make the drop on the feeders smaller as we approach the power house, we shall have a continual gradient in the rails all the way to the station, thus utilizing the conductivity of the tracks to any desired extent. Insulated-feeder systems embrace a number of modifications, chief among which are the following:

(1) Boosters in Separate Feeders—A method used in Europe is that of inserting a booster in each feeder or group of feeders. By varying the voltage of the individual boosters, almost any desired potential conditions can be obtained in the track network. To offset this advantage is the large first and depreciation and operation cost of the boosters. The copper feeders can be designed solely from the point of view of economy and thus a much more economical installation can be obtained than where the drop on the cable must be no greater than the drop on the rails.

(2) Insulated Negative Feeders Without Boosters—The potential necessary to force the required current through the negative feeders without the use of additional copper is obtained by removing the direct tie between negative bus and rails at the power house and substituting in its place a properly designed resistance tap.

In designing the feeders a study is made of the load distribution over the entire territory; the most natural points for taking off current are selected and the number of amperes to be taken off at each point determined. A preliminary value of potential drop on the first feeder is then assumed; from this drop and the current to be carried by the feeder, together with its length, the cross-section of the feeder must be designed. Proceed in a precisely similar manner for the other feeders and the resistance taps near the power house. In case the calculated feeder should be too small to carry the required current without overheating, it must be made

sufficiently large to carry the current and an additional resistance inserted, preferably at the power house, to give the necessary voltage drop.

To determine whether the original assumption made in regard to voltage drop on the first feeder was the one that would give approximately the most economical installation, sum up the total cost of the feeders installed, and determine the proper annual charge, including interest, taxes and depreciation, and also calculate the total annual value of the energy lost in the feeders and resistance tap. If these are approximately equal, the voltage drop assumed was the proper one.

Against the decided advantages of the resistance-tap over the booster method must be set the objection that the former is less flexible than the latter, and cannot be made to respond as readily to meet the exigency of shifting loads. The really important consideration is to take care of average normal conditions, and this the system will do automatically if properly designed. A large measure of flexibility can be imparted to the system by varying the resistance of the individual feeders at the power house. Since the voltage drop on any feeder will, as a rule, be of the order of ten or more times the voltage drop on the rails between adjacent feeder taps a change of 10% or less in the resistance of a feeder or group of feeders would take care of a shifting of the load tending to produce local variations of 100% in the rail currents.

(3) Single Booster Scheme—This method is designed to eliminate the disadvantages of the insulated negative-feeder scheme with and without boosters. All of the negative feeders except the power-house tap are brought to a feeder bus and a single large booster is connected between this bus and the negative bus of the generators. Proper distribution of current between feeders is secured by proportioning their resistances. The power-house tap is led through the field of the booster. In consequence of this the voltage tending to force current through the feeders is proportional to the current in the power-house tap and hence the proper division of current between the two sections is automatic.

(4) Inverted Booster Scheme—This differs from the method without boosters only in the substitution for the resistance tap of an "inverted booster"; that is, a booster so arranged as to produce a counter e.m.f. in the power-house tap sufficient to cause the proper amount of current to flow over the insulated feeders. This inverted booster consists of a series motor in the tap circuit coupled to a practically constant-speed generator feeding some external circuit. The counter e.m.f. of the series motor gives the drop required on the feeders, and the power consumed by the motor in excess of the losses is returned to the system through the generator end of the unit. The counter e.m.f. of the machine may easily be made practically proportional to the current taken from the tracks by the motor, and the drop on the feeders and consequently the current taken by them will be nearly proportional to the motor current, which is approximately the condition desired. The operation is, therefore, entirely automatic for normal load conditions.

NUMBER AND LOCATION OF POWER HOUSES AND SUBSTATIONS

The effect of the number of feeding points on the potential drops in the rails and the consequent leakage of current from the tracks is obvious but the economic aspect is complex. As the number of stations is reduced the capacity of each must be increased, with the result that the current flow and potential gradients in the rails approaching the power house will be greater. It can be shown that the current picked up by the pipes may increase as the second, third, or even higher power of the distance between drainage points, according to the character of the system. Any increase in the number of such points will reduce in much greater degree the flow of stray currents in the pipes. The number of drainage points can be increased to any extent desired by the proper use of insulated negative feeders but the fewer the stations the longer and heavier the feeders must be. An increase in the number of stations may often prove to be a pronounced economy when we consider also the distribution of the power on the positive side as well as its return on the negative.

Calculation of the value of lost power is by no means simple. It is not sufficient simply to determine the total energy loss during any given time and multiply this by the cost of power in order to determine its value. We must consider that the loss of power is proportional to the square of the load and hence is greatest at time of peak load when the capacity of the power station is usually taxed to its utmost. The capacity of the generating plant and hence the fixed charges on the cost of power are thereby increased; or, if the power be purchased, there is usually a fixed charge imposed on the maximum demand. In any case if the operation of the railway system is such that the line losses give rise to an increased demand for power at time of peak load, the cost of the lost power will be greater than the cost of the power utilized at the cars.

We have to consider also the effect of this drop of potential on the character and cost of the car service. Low voltage means lower average car speed, with the consequent increase in the number of cars required to operate at a given headway, which in turn increases both fixed charges and operating costs. Any change in the distribution system, therefore, whether a change in the number of stations or any other change which affects materially the line losses, will exert a marked effect on the cost of operation of the system.

In designing a method of electrolysis mitigation, therefore, many things have to be taken into account quite apart from the technical points regarding the electrical condition of the negative return if a proper balance is to be maintained between the cost of making the proposed changes and the benefits resulting therefrom.

Investigation of Concrete Drain Tile in Alkali Regions

In the latter part of 1913, an advisory committee composed of members of various Federal bureaus and of the Association of American Portland Cement Manufacturers initiated a series of tests looking into the behavior of concrete drain tile in the alkali-bearing waters of the West. The general subject of the disintegration of concrete as a result of the action of the salts in such alkalis has been investigated a number of times, but never in a thorough manner. Examinations have been made of existing structures, whose history was at best vague, as well as studies of the action of alkali on laboratory specimens, but the combination of a prearranged manufacture with field conditions of service has not existed. Such a combination this committee undertook, under the specifications noted below. The committee consisted of F. W. Hanna, U. S. Reclamation Service; S. H. McCrary, Drainage Engineer, U. S. Department of Agriculture; R. J. Wig, U. S. Bureau of Standards and L. R. Ferguson, Association of American Portland Cement Manufacturers. The tile have been laid and their condition will be examined and reported on from time to time.

PROGRAM OF PROPOSED FIELD INVESTIGATIONS OF THE ACTION OF THE SALTS PRESENT IN ALKALI WATERS ON CEMENT AND CONCRETE

TYPE OF TEST PIECE—The test piece is to be made in the form of a standard agriculture concrete drainage tile 8 in. in diameter, except that the hand-tamped tile are to have a wall thickness of approximately 1½ in.

PLACE OF MANUFACTURE—All tile are to be manufactured at a concrete tile plant which is supplied with good aggregate and fully equipped to manufacture a good grade of tile.

MIXTURES TO BE TESTED—Five hundred and fifty tile are to be made of each of the following mixtures. All concrete is to be machine mixed:

1. Concrete in the proportion of one part portland cement to 2½ parts best local clean aggregate to be made of a consistency such that the forms may be removed immediately after moldnig. The tile are to be hand-tamped and cured by sprinkling with water.

2. Concrete in the proportion of one part portland cement to 2½ parts best local clean aggregate to be made of a quaking consistency, the forms to be left on for 12 to 15 hr. after molding. The tile are to be hand-tamped and cured by sprinkling with water.

3. Concrete in the proportion of one part portland cement to two parts of graded aggregate to be made of as wet a consistency as will permit of an immediate removal of the forms. The tile are to be hand-tamped and cured in a steam vapor.

4. Another set of tile are to be made and cured the same as mixture 3 and are to be coated with cement grout within 24 hr. after molding.

5. Another set of tile are to be made and cured the same as mixture 3 and are to be coated with an asphalt or tar after the tile are six weeks old.

6. Another set of tile are to be made and cured the same as mixture 3 except ferrous sulphate is to be added to the water used in mixing the concrete from which the tile are to be made.

7. Concrete in the proportion of one part portland cement to three parts of best local clean aggregate to be made of a consistency such that the forms may be removed immediately. The tile are to be made by machine and cured by steam vapor.

8. Concrete in the proportion of one part portland cement to four parts of best local clean aggregate to be made of a consistency such that the forms may be removed immediately. The tile are to be made by machine and cured by steam vapor.

9. Concrete in the proportion of one part portland cement to 1½ parts of graded aggregate to be made of a consistency such that the forms may be removed immediately. The tile are to be made by machine and cured by sprinkling with water.

10. Concrete in the proportion of one part portland cement to 1½ parts of graded aggregate to be made of a consistency such that the forms may be removed immediately. The tile are to be made by machine and cured in a steam vapor.

11. Concrete in the proportion of one part portland cement to three parts of graded aggregate to be made of a consistency such that the forms may be removed immediately. The tile are to be made by machine and cured by sprinkling with water.

12. Concrete in the proportion of one part portland cement to four parts of graded aggregate to be made of a consistency such that the forms may be removed immediately. The tile are to be made by machine and cured by sprinkling with water.

13. Concrete in the proportion of one part portland cement to four parts of graded aggregate to be made of a consistency such that the forms may be removed immediately. The tile are to be made by machine and cured in a steam vapor.

14. Concrete in the proportion of one part portland cement to three parts graded aggregate to be mixed to a creamy consistency. The tile are to be hand made by the wet process and cured by sprinkling with water.

15. Concrete in the proportion of one part silica cement to three parts of graded aggregate to be mixed to a creamy consistency. The tile are to be hand made by the wet process and cured by sprinkling with water.

PLACE OF EXPOSURE—Forty tile of each kind are to be shipped to each of the following locations:

1. Uncompagre Project, Colorado.
2. Sun River Project, Montana.
3. Shoshone Project, Wyoming.
4. Yuma Project, Arizona.
5. North Yakima Project, Washington.
6. Pecos Valley (Roswell), New Mexico.
7. Grand Junction, Colorado.
8. Huntington, Utah.
9. Columbia, Missouri.
10. Northern Minnesota.
11. To convenient storage point.

METHOD OF EXPOSURE—The tile are to be placed in running drains to be subjected to the most severe conditions available in the above districts. The drains are to be installed by the engineers of the Reclamation Service and drainage division of the Department of Agriculture.

Twenty-five lengths of each class of tile are to be placed adjoining. At the upper end of the drainage line there are to be 10 sections, 14 ft. in length, in which there will be only one tile of each class in each section.

TESTS—At the end of each year after the tile have been placed the drainage line is to be inspected and at least one section at the upper end is to be uncovered and removed from the line to be tested in a tile-testing machine. Complete analyses are to be made of all classes of tile showing any indications of failure.

The soils surrounding the tile and also samples of water running through the drain are to be analyzed at the time of high and low water plain excepting in the case of the tile drains exposed in fresh water in Minnesota and Missouri.

One set of all classes of tile is to be exposed to the atmosphere protected from the elements to be tested yearly for comparison with the tiles placed in the drains.

GENERAL CONDITIONS—All tile are to be manufactured and tests conducted under the supervision of the Bureau of Standards.

One brand of portland cement is to be used for the manufacture of all tile excepting those to be made with silica cement.

Complete chemical and physical tests are to be made of all materials used.

Complete records are to be kept of each step in the process of manufacture, exposure and testing.

Memorial to Sir William H. White—It is proposed to erect a memorial to the late Sir William H. White, the famous English engineer and naval architect. The movement is being started by English engineers, but as Sir William was well known in this country and was an honorary member of both the American Society of Civil Engineers and the American Society of Mechanical Engineers, it is expected that many in this country will be glad to participate. Those desiring to do so should communicate with the secretary of one of these societies.

Recommendations for Reinforced-Concrete Highway Bridges and Culverts*

The following recommendations are submitted to the Institute, not as final, nor even as voicing the unanimous opinion of the members of the Committee, but merely as preliminary suggestions, in the hope of bringing out the discussion of doubtful points and information regarding questions which the Committee could not answer.

DEAD LOADS

The following are given as average weights per cubic foot of the materials mentioned, but if the weights of the materials to be actually used are definitely known to be different from those given, the correct weights should be used.

Materials	Lb. per cu.ft.
Earth filling	110
Plain concrete	150
Reinforced concrete	155
Steel	490
Cast iron	450
Vitrified brick	140
Common brick	125
Granite and limestone masonry	165
Sandstone	160
Macadam-Telford	150
Pine, fir, etc.	42
Oak and yellow pine	48
Creosoted timber	60

The weight imposed by earth filling should, in ordinary cases, be considered as including all filling included between vertical planes passing through the faces of the abutments.

If, however, the height of the fill exceeds about two-thirds the distance face to face of the abutments, the live load may be neglected and a very considerable proportion of the weight of the filling considered will be supported by friction between it and the approach filling. The amount of load thus transmitted is greatly affected by the cohesion of the soil, of which there is nearly always more or less.†

This whole subject requires further investigation.

LIVE LOADS

CLASS "A" BRIDGES—Main thoroughfares leading from large towns.

In view of the extensive introduction of the heavy motor trucks and traction engines, and the probable general use of such vehicles in the future, it is recommended that bridges on main thoroughfares and other roads which are likely to be used for heavy hauling, be designed to carry 20-ton trucks, with axles about 10 ft. c. to c., 14 tons on rear axle and 6 tons on fore axle; wheels about 5 ft. c. to c. Outside of the large cities it is recommended that only one such vehicle be assumed to be on the bridge at any one time, the liklihood of more than one being on the bridge, in a position to produce maximum stresses at the same time, is so remote that this assumption is considered safe. It is advised that such very heavy loads be considered as occupying only the ordinary width of the road, about 8 ft. in width and about 35 ft. in length. Congested traffic of heavily loaded wagons or motor trucks will rarely impose a load of more than 100 lb. per sq.ft., over a considerable area. The above mentioned 20-ton truck gives a load of about 140 lb. per sq.ft., on the area actually occupied, but it is considered extravagant to assume that a large bridge is covered with such heavy loads. One hundred pounds per square foot is thought ample to assume for the loading of spans more than 60 ft. long, in designing the trusses or main girders. It is thought to be safe to reduce this assumed load in the case of longer spans, to the following amounts:

Lenth of span, ft.	Assumed load lb. per sq. ft.
80	90
100	80
125	75
200 and over	70

With all intermediate spans in proportion.

The greatest load that is liable to be imposed on a bridge sidewalk, occurs when there is some excitement in the neighborhood, which attracts a large crowd, and for which the bridge affords an especially good point of view. In that case the crowd forms a compact mass, against the railing, not more than 4 ft. deep, making a load seldom exceeding 100 lb. per sq.ft., over a very considerable space. The remaining portion of the sidewalk may be covered by a moving crowd which can scarcely weigh more than 40 lb. per sq.ft. It may be advisable, sometimes, to so design sidewalk slabs, that if

*Report of Committee on Reinforced Concrete Highway Bridges and Culverts, American Concrete Institute presented at Annual Convention, Chicago, Ill., Feb. 17, 1914.

†This paragraph is not approved by Mr. Lemuel Holmes.

a street car or motor truck accidentally gets upon the sidewalk, it will not go through. Such accidents are so rare, that it is thought safe to allow materials to be stressed somewhat beyond the elastic limit in such cases.

CLASS "B" BRIDGES—Although it is impossible to determine beforehand, especially in the newer parts of the country, whether any given road is to be used for heavy traffic, it seems extravagant, at least in the cases of larger spans, to design bridges to carry much heavier loads than can be expected to come upon them. It is recommended that bridges of this class be designed to carry 15-ton trucks, with axles 10 ft. apart, 5 tons on the front and 10 tons on the rear axle. This will allow for a considerable overloading of existing motor trucks. It is further recommended, that only one truck be assumed to be on the bridge at one time, in designing the floor system, that it be assumed to cover a width of 8 ft. and a length of 35 ft. and that the remainder of the bridge is covered with a load of about 90 lb. per sq.ft., for spans up to 60 ft.

The longer spans, the trusses and main girders should be designed for the following loads:

Lenth of span, ft.	Assumed load lb. per sq.ft.
80	80
100	70
125	65
150	60
200 and over	55

With intermediate spans in proportion.

Sidewalks should be designed to carry the same loads as in the case of Class "A" Bridges.

SPECIAL BRIDGES—City bridges and bridges carrying traffic connected with mines, quarries, lumber regions, mills, manufactories, etc., require special consideration and should, of course, be designed to carry any load which can reasonably be expected to pass over them, bearing in mind the likelihood of heavy traction engines and motor trucks coming into extensive use in the not distant future.

STRINGER LOADING—The maximum stress in a stringer, due to a wheel load, occurs evidently when the wheel is directly over it. It is not thought proper to assume any distribution of the load to adjacent stringers, unless the bottom reinforcement in the slab is made continuous. In that case the distribution is proportional to the relative stiffness of the slab and the stringers, said stiffnesses being proportional to the moments of inertia, multiplied by the modulus of elasticity of materials and inversely proportional to the cube of the span. In determining this distribution, due account must be taken of the fact that deflection of the slab decreases toward the end of the stringers, and also of the fact, that whatever load is carried to the adjacent stringers, deflects them also. It is therefore recommended that wherever practicable the bottom reinforcement of slabs be made continuous over the stringers.

SLAB LOADING—The distribution in a direction parallel to the supports of a concentrated load resting on a slab, supported at two opposite edges only, evidently depends upon the same principles as those mentioned under "Stringer Loading." The main difference being that what corresponds to the stringer in the former case is of indefinite width in the present case. Adequate theoretical investigations of this question appear to be lacking. For the present it seems fair to assume that the distribution each side of a concentrated load is equal to about one-third the length of the span, and that the cross reinforcement should be designed accordingly, which would require it to have an area of at least one-half of the principal reinforcement. The distribution of a concentrated load through earth filling on the top of a slab, does not appear to be very well understood.

BRIDGES CARRYING ELECTRIC CARS—Electric traction is still in its infancy and nobody is able to forecast its future development. It seems probable, however, that it will not be profitable to run cars weighing more than 50 tons each, at a speed that would be permitted on any public road. If very high speeds are desired, the traction company will doubtless be required to operate over its own right-of-way. It is recommended that bridges carrying either urban or interurban electric cars be designed to carry 50-ton cars on two trucks, spaced 30 ft. c. to c., each truck having two axles spaced 7 ft. c. to c. The Committee sees no reason for changing the customary practice of assuming that an axle load is distributed over 3 ties.

LOADING ON ARCHES—The deflection of an arch being much less than that of a beam of the same length, the method recommended for determining the lateral distribution of a concentrated load over arch sheeting appears to be different from the distribution over flat slabs. It seems doubtful if the distribution in each direction can be greater than twice the thickness of the arch sheeting. This question should be investigated.

STRESSES IN ARCHES—As all arches that are not provided with hinges act as elastic arches until cracks are formed, due to excessive tension at some point of joints in the concrete, it is manifestly proper to calculate the stresses in them according to the elastic theory.

As concrete is a very poor conductor of heat, it is not thought necessary in calculating reinforced-concrete arches, to assume so much variation in temperature as is often designing steel structures, although the outside layers of concrete are of nearly the same temperature as the surrounding air, and those layers are stressed more heavily than any of the others, it is thought that an extreme variation of about 80° F. in the Northern States is sufficient to allow for in any case, and that can be reduced if the arch ring is thicker or if there is much earth filling in the spandrels.

BEARING POWER OF PILES

The usual formula for the safe bearing power of wooden piles is:

$$R = \frac{2WH}{S + 1}$$

in which,

W = Weight of hammer in pounds,

H = Height of fall of hammer in feet,

S = Penetration in inches per blow, average of last few blows.

If reinforced-concrete piles are molded before being driven, the head of the pile is usually cushioned more or less, and the pile is generally much heavier in proportion to the weight of the hammer than is the case with wooden piles. It is recommended that the hammer should be at least as heavy as the pile.

If concrete piles are molded in place, measures should be taken to prevent damage to them by the driving of neighboring piles or otherwise.

BEARING POWER OF SOILS

This subject is under an investigation by a committee from the American Society of Civil Engineers. It may be some years before their final report is submitted and even that will be subject to revision from time to time, as human knowledge is extended. In the meantime the Committee would submit the following preliminary table:

Material	Safe bearing power, tons per sq.ft.
Quicksands and wet soils	0.1 to 1.0
Dry earth, according to depth below surface	1 to 3
Moderately dry clay confined	2 to 4
Dry stiff clay	4 to 6
Sand confined	2 to 6
Sand compact and cemented	4 to 8
Gravel cemented	8 to 12
Rock	25 to 200

Foundations should be carried down below frost unless they are on rock and thoroughly drained. Soil that contains the roots of plants is generally compressible. Undisturbed soil is much less compressible than filling or similar soil, even though it has been in place many years. The bearing power of sand, gravel and dry clay increases rapidly with the depth below the surface of the ground. None but the smallest structures should be founded on earth filling.

The pressure of earth against abutments, wingwalls and retaining walls varies so widely with the character and condition of the earth in question, that nothing more than a few general suggestions can be given. It is hoped that the Committee from the Society of Civil Engineers, which is studying the bearing power of soils, will take up this subject also. In ordinary cases where the filling is well drained its pressure will seldom be more than that of a liquid weighing 25 lb. per cu.ft., and is frequently much less. The pressure due to land slides is often several times this amount. If the filling is clayey and is allowed to become waterlogged, its lateral pressure is greatly increased. The effect of the freezing of the filling must also be considered.

REQUIRED WATERWAY

The usual formula for waterway for culverts is:

$$a = CA^{\frac{1}{2}}$$

in which,

a = Required area of culvert in sq.ft.

A = Drainage area in acres,

C = A constant, depending on the length and character of the drainage area and may vary from about 0.3 to 2.0 in regions where the mean annual rainfall is about 50 in.

The capacity of the culvert will be much greater if the wingwalls are flush with the abutments, and flare about 30°, the sides and bottom of the culvert are smooth and straight, and the bottom has a good slope.

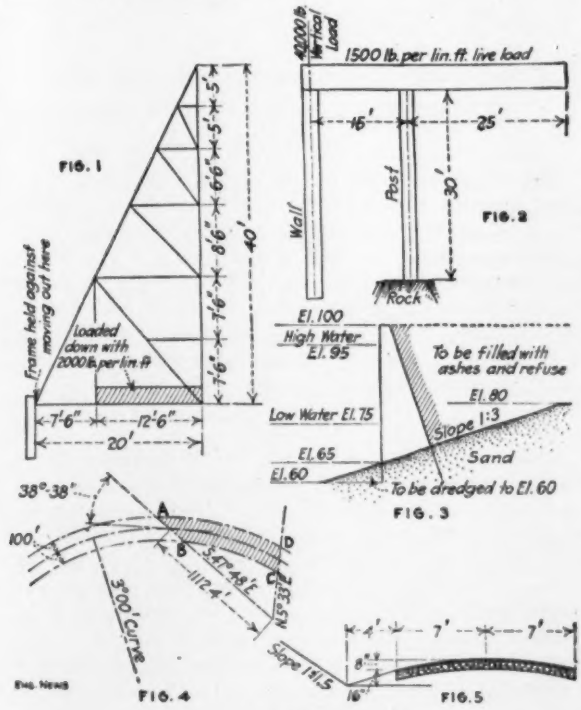
If reliable information covering a number of years can

be obtained regarding the adequacy of the old bridge crossing the same stream, it is much more useful in determining the size of a culvert or bridge than any formula.

WILLIS WHITED, Chairman.
LEMUEL HOLMES.
A. N. JOHNSON.
A. M. LOVIS.
HENRY H. QUIMBY.

New York State Civil Service Examination Questions for Assistant Civil Engineer

In accordance with a recent request for information regarding the qualifications demanded in an assistant civil engineer, we publish the following questions used in a New York State civil service examination held on Jan. 17, 1914. The examination consisted of two parts and competitors were required to take three exercises of Part I, counting 20 points each, and four of Part II, making up a weight of 50 for this part. Should the conditions of a problem seem insufficient, other data may be assumed (consistent with the data given). Reference books were permitted in the first part, but not in the second.



PART I

1. The form for the face of a concrete wall 40 ft. high consists of a triangular frame shown on Fig. 1, loaded and held as indicated. Assuming that the concrete is placed in 10-ft. lifts or layers and that its lateral pressure is equal to that of a liquid weighing 100 lb. per cu.ft.; find the maximum stress in each piece of the frame. **Weight 20.**

2. Fig. 2 represents a reinforced-concrete post and girder which may be subject to a live uniform load of 1500 lb. per lin.ft. on all or any part of its length. It is assumed that the inner end of the girder is not rigidly fixed, and that the post rests upon bedrock. Make a detailed design of the post and girder, computing unit stresses throughout. **Weight 20.**

3. It is required to build a concrete dock wall in the position and for the conditions indicated on Fig. 3.

Design a section for the wall, computing the maximum pressures on it and its footing both at high and at low water. **Weight 20.**

4. Given the following data for the design of a concrete box culvert 70 ft. long; the stream has a hard clay bed 10 ft. wide; gravel banks rise on a 1 on 3 slope; high-water mark shows a depth of 3 ft. These figures apply to a stretch whose slope is 1 on 800.

The culvert is to be designed to pass the high-water discharge without flowing under a head; and, in addition, it is required to pass 500 ft. per sec. when flowing under a head not to exceed 8 ft. **Weight 20.**

5. From the data given in Fig. 4 compute the area of the figure ABCD. **Weight 20.**

PART II

6. Describe the surveys that should be made, giving the makeup of the party, the method used, and the information to be obtained, preliminary to the construction of:

(a) A macadam highway, built partly on the line of an old road and partly on new location.

(b) A large sewer, running through the main street of a large city, partly in the center of the roadway, and partly under the sidewalk. **Weight 15**

7. Under what circumstances would an engineer in local charge of a contract be justified in ordering a contractor to discontinue? What effect would such action have in extending the date for completion?

What should be the duties of an inspector and what matters should he note particularly, if assigned to look after the excavation and concreting of a cutoff wall under a dam? **Weight 15.**

8. Assume that the road section (Fig. 5) is made of a 2-in. bituminous macadam top course, consisting of 1½-in. stone bound with asphalt, of which 1½ gal. is used per sq.yd., and a 5-in. bottom course of crusher-run stone passing a 3-in. screen, rolled as delivered without binder. This road is to be built along a clay hillside and a rocky ridge. Assume first that the road carries only a light farm and automobile traffic; and then as a second case, assume it to have a heavy interurban travel of every description.

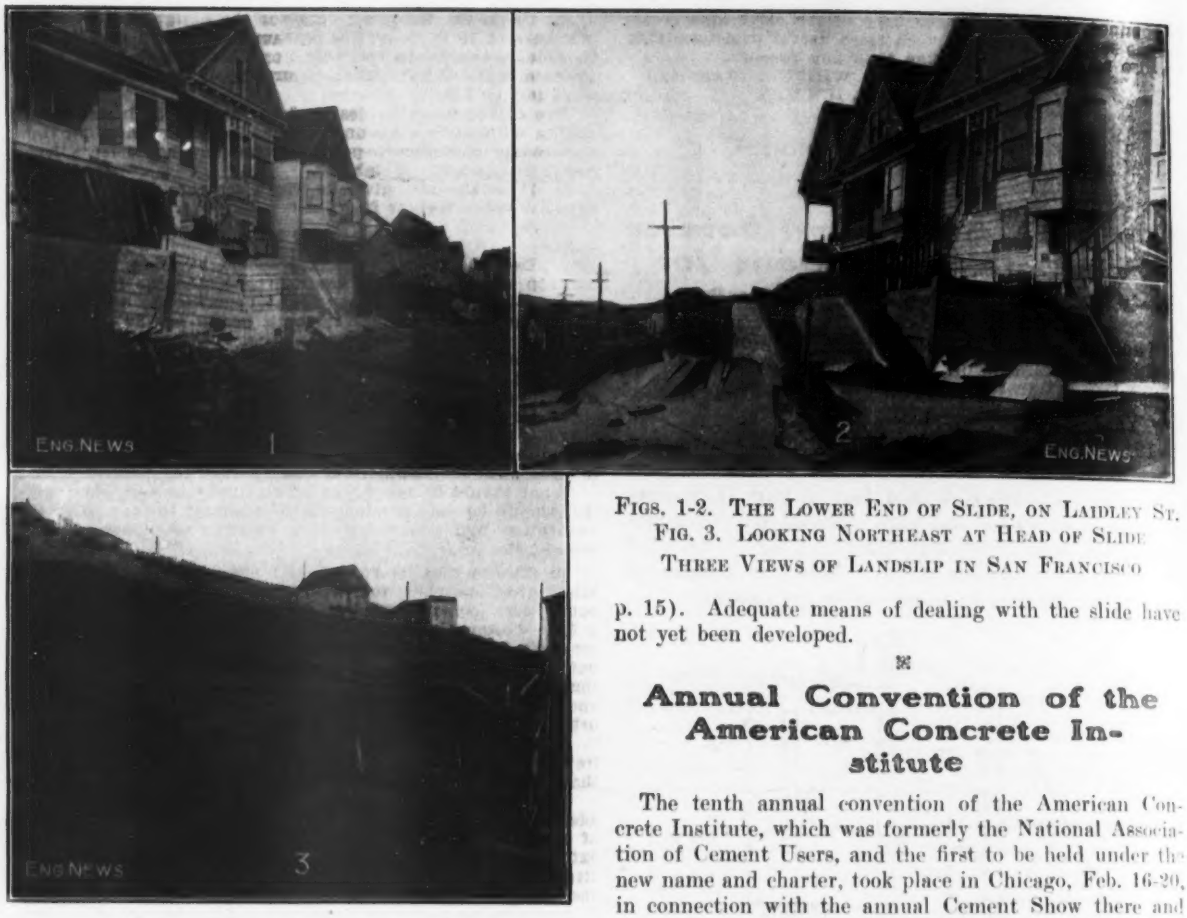
Criticize the section given, pointing out and explaining its defects and how it could be improved and give sections that you consider best adapted to the conditions. **Weight 15.**

9. Describe the type of foundation that would be best adapted to support a high building resting upon (a) a bed of dry sand, 40 ft. deep, overlying hard clay; (b) wet clay, extending to rock at a depth of 50 ft.; (c) wet sand in a stratum 30 ft. thick, followed by 100 ft. of clay, overlying a shale rock. **Weight 10.**

10. Suppose you were appointed to have charge of a large reinforced-concrete sewer job that had been about half completed. Describe the information that would be needed to prepare your first monthly payment, and how it would be gathered, and give the form in which the estimate would be made up. **Weight 10.**

11. Give the ultimate strength and allowed stress per square inch in tension, compression, shear, and bending (extreme fiber) of the following: (1) Medium structural steel; (2) cast iron (for water pipe); (3) portland-cement mortar 1:2 mixture, age one month; (4) yellow pine (in large pieces); (5) common brick; (6) limestone (for concrete). Describe the common faults or defects that may cause failure in the use of each of the materials mentioned. **Weight 10.**

Heavy Rainfall; Landslide Dams Stream—In July, 1911, the terrific storm which swept over the Island of Luzon seemed to reach its height in the vicinity of Baguio and the Benguet Road. In one day 38.8 in. of water fell, and during the four worst days the rainfall totaled 66.45 in. This unprecedented precipitation on the bare sides of the cañon of the Bued River, into the walls of which the Benguet Road for a considerable portion of its length is cut, resulted in great damage to the lower half of the highway. Two enormous slides, the contents of which were estimated at over 1½ million cu.m., occurred at kilometer 13, forming a natural dam 150 ft. high and impounding the torrent to that height. During the few hours of the existence of this dam, gravel was deposited for a distance of several kilometers above it to a height of 40 and 50 ft. above the former river bed. Its bursting released a tremendous flood which carried everything before it. The temporary replacement of the old road was made much more feasible in September, when another heavy rainfall of 31 in. in 24 hr. cut through the vast deposit of gravel and boulders which obliterated the old location and swept a large part of it away. The work of reopening the road was started in October and vigorously carried on until Dec. 22, 1911, when automobile trucks were again able to travel its entire length. This work cost about $50,000. [From 1912 report of Secretary of Commerce and Police, Manila.]



FIGS. 1-2. THE LOWER END OF SLIDE, ON LAIDLEY ST.
FIG. 3. LOOKING NORTHEAST AT HEAD OF SLIDE
THREE VIEWS OF LANDSLIP IN SAN FRANCISCO

Earth Slide in San Francisco

What an earth slide did to a street and four houses in San Francisco a few weeks ago is shown by the three photographs herewith, by W. L. Huber, Consulting Engineer. The lay of the land can be understood with the help of the plan sketch Fig. 4 (approximately to scale). The damage done is worst at the lower or Laidley St. end of the slide, where the pavement is buckled up and destroyed and the house fronts and entrances wrecked. The break at the upper end of the slide is clearly shown in Fig. 3.

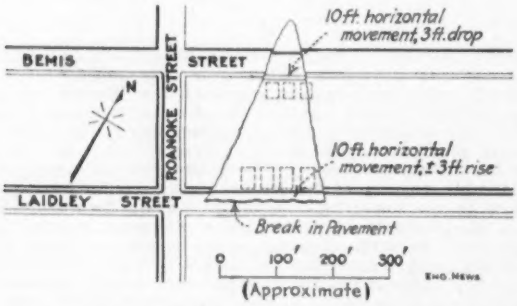


FIG. 4. SKETCH MAP OF LAIDLEY ST. SLIDE, SAN FRANCISCO

This is a hillside slip, closely similar to the one at Bellevue, Penn. (ENGINEERING NEWS, Jan. 1, 1914, p. 15). Adequate means of dealing with the slide have not yet been developed.

Annual Convention of the American Concrete Institute

The tenth annual convention of the American Concrete Institute, which was formerly the National Association of Cement Users, and the first to be held under the new name and charter, took place in Chicago, Feb. 16-20, in connection with the annual Cement Show there and the convocation of a number of allied interests. About 200 members and guests were registered. Eight regularly scheduled sessions and one extra session were held and a banquet was given on one evening.

CEMENT

The one paper on cement was by P. H. Bates (U. S. Bureau of Standards) and was entitled "The Properties of Portland Cement Containing a High Percentage of Magnesia." The current specifications for portland cement all contain a clause limiting the magnesia content in cement to 4%. According to Mr. Bates this stipulation is based on a minority report of a committee of the German cement manufacturers made some years ago. This minority report held that a content of magnesia over 4% tended to give poor concrete and in a concrete made from a cement with 8% or more magnesia, cracking was produced. At the same time a majority report, apparently since disregarded, held that as high as 10% magnesia would not be detrimental. In investigation of the subject the experimental rotary kilns of the Bureau of Standards at Pittsburgh were used and a cement clinker produced in which the magnesia content ran as high as 19%. This clinker was made by burning a high magnesia rock and not by the addition of magnesia to cement already burned.

It was found that up to about 7% magnesia no new compounds other than those ordinarily found in portland cement were formed, but that at 9½% magnesia the mineral known, when found in nature, as monticellite

($MgO\,CaO\,SiO_2$) was formed and at 14%, spinel ($MgO\,Al_2O_3$) was formed. The specific gravity was not affected nor was there any free magnesia.

Strength tests on this compound showed that the magnesia increases the strength, both tensile and compressive, up to 5% content, but produces a slower set. All of the pieces passed the tensile test but all failed in compression after being placed in the autoclave.

The general preliminary deduction is that within certain limits, higher than those now allowed, magnesia is not a detriment to cement clinker, and in any event it does not occur uncombined. Its presence in excess of 4% is certainly not an incentive to the deterioration of concrete in sea water.

Tests

Two papers on tests were read, one by Burtis S. Brown (Consulting Engineer, Boston, Mass.) and the other by W. A. Slater (Engineering Experiment Station, University of Illinois). Both papers described extensometer measurement tests on girderless floor systems. Mr. Brown first made brief references to the technique of such testing and then described the method of conducting a test on a special one-story floor erected for the purpose of test at the Worcester Polytechnic Institute. This floor was 57x57 ft. in plan and was carried on 25 columns, spaced about 14 ft. c. to c. This gave 9 interior, 12 wall and 4 corner columns, all provided with spread caps on which the floor was laid. Different types of girderless floor construction were used in different sections of the slab so that a comparative estimate could be made. Extensometer measurements were made at critical points. Mr. Brown did not give any of the details of the results obtained, stating that they were to be published in a later paper.

Mr. Slater's paper described in detail the results of a similar test on a panel of the girderless floor of the Shredded Wheat Co.'s building at Niagara Falls, N. Y. His own abstract of the paper is as follows:

The building is of reinforced-concrete construction of the flat slab type, is three stories in height, and is divided by columns into panels 20 ft. by 22 ft. The test was made on the first floor above the basement story. This floor is 7 in. thick in the central portion of the panel, is 9 in. thick throughout the area of 8 ft. 6 in. square surrounding each column, and is designed to carry 125 lb. per sq.ft. live load. The columns are octagonal in shape, those of the basement story being 25 in. in the short diameter and ending at the top in 3-ft. 6-in. flared heads. The columns of the story above are 23 in. in the short diameter. The reinforcement, all of which is placed in the two directions parallel to the sides of the column, is designed to resist positive moment at the centers of all spans and negative moment at the columns and across the edges of all panels.

Originally nine panels were loaded to 191 lb. per sq.ft., after which the load was removed from six panels, leaving three adjacent central panels loaded. These three panels lie in a row and are one span removed from the exterior wall of the building. Finally, the load was removed from all but one panel, the central panel of the entire test area and the load on this panel was increased to 243 lb. per sq.ft.

Deformations were measured at 288 gage lines and the results were very consistent among themselves. It was found that the condition of the three adjacent panels loaded gave steel stresses at the center of the span more than 50% greater than with nine panels loaded, and that the stress at the support was about 20% less with three panels loaded than with nine panels loaded. It seems apparent that the tendency of the floor to deform as a cylinder instead of as a saucer, its action as a continuous beam, and lateral distribution of stress to unloaded portions of the slab all contributed to these differences betwen the stresses under nine-panel load and those under three-panel load. The results of the test seem to check the conclusion arrived at analytically that the greater portion of the unbalanced moment was carried by columns and very little by the slab. This conclusion is somewhat in conflict with the observation that there was so great an increase in the stress at the center of the panel when only three panels were loaded. In this respect it is typical of the generally observed fact that the results obtained from such tests are usually indicative of the general trend of results, but cannot be applied too rigidly as final conclusions.

The Committee on Specifications and Methods of Tests for Concrete Materials presented a long and very valuable report for the most part based on the results of a series of tests planned by the committee and executed by the laboratories of several universities in various parts of the country. These tests had as an object the determination of the size and shape of a proper standard concrete test piece for field and laboratory tests and also the relative strength of concrete test pieces of different shape and size for comparison with the strength of test pieces of the standard shape. The general deductions of the committee were as follows:

The best shape of test piece is a cylinder. A height of double the diameter presents advantages over other proportions. Tests of similar cylinders show slightly less variation from each other than tests of square prisms or cubes. Where the height is double the diameter, the effect upon the strength of slight inequalities in height of different test pieces is eliminated.

Cubes, cylinders or prisms, not shorter in height than their least diameter, can be used for comparative tests. For comparison with test pieces of standard shape, the strength of these may be corrected by a ratio.

The smallest dimension of the test piece should be at least four times the size of the coarsest particles of stone.

The shape of the test piece appreciably affects the strength.

The strength of test pieces increases as the height decreases. Assuming a test piece whose height is double the width as a basis for comparison, a cube of the same mix will have about 37% greater strength and a test piece whose height is half its width, about 109% greater strength. As the length of the test piece becomes greater than two times the width, the strength decreases slightly.

Tests of similar concrete made with different percentages of water tend to confirm conclusions previously established, that an increase in quantity of water in mixing reduces the strength of the concrete to a marked degree, and that this reduction is especially marked at the earlier ages.

The growth in strength with age, using different consistencies of concrete can be estimated.

The higher loads carried by test pieces which are shorter than one and one-half diameters are ordinarily attributed chiefly to the effect of the restraint against lateral swelling of the test pieces under load, which is developed by the friction between the bearing plates of the testing machine and the ends of the test piece.

Researches in literature and tests by individual members of the Committee indicate that the strength of concrete of given proportions increases as the size of the coarse aggregate increases. In the present series the results on this point are inconclusive, although tending in this direction.

The size of the test piece appears to affect the strength to a certain degree, but because of varying results in different laboratories, no definite conclusion has been drawn.

The method of storing the test pieces has a marked influence upon the strength of the concrete. Further tests on this point are necessary before drawing definite conclusions as to relative strength under different conditions.

It is recommended tentatively that the shape of the standard concrete test piece be a cylinder having a length of twice the diameter. The diameter should not be less than 6 in. except with very fine stone, when 4 in. may be adopted. It appears advisable that the diameters of the cylinder should be at least four times the maximum size of the coarsest particles of the aggregate. The 8- by 16-in. cylinder, which has been adopted by the Joint Committee on Concrete and Reinforced Concrete is a satisfactory size when shipment of materials or other limiting condition does not preclude its use.

The strength of cubes or cylinders having a length equivalent to the diameter can be approximately converted to strength of cylinders of double the diameter by multiplying by 0.73.

For field tests of concrete in actual construction, concrete should be taken from the loose mass after it is dumped from

the barrow or other receptacle and before it is tamped or otherwise compacted.

The size of field test pieces should conform to the recommendations given above.

Field test pieces should be embedded completely in sand located near the site of the structure and the sand kept continually moist until ready to ship to the laboratory. The test pieces for shipment should also be packed so as to retain their moisture.

The tables given by the committee are very valuable and will be reproduced later in ENGINEERING NEWS.

CONCRETE TILE

One session was devoted to the discussion of the concrete tile for drainage purposes. At this session Dean A. Marston (Iowa Engineering Experiment Station, Ames, Iowa) explained the work of the Committee on Drain Tile of the American Society for Testing Materials, of which committee he is chairman. This committee is now engaged in formulating specifications for drain tile and has made a number of experiments looking into their general strength and absorption qualities. Up to this time such tile have not been designed nor laid according to any very good or stable engineering standards but it is hoped that the present study will lead to specifications which will materially increase their efficiency.

Rudolph J. Wig (U. S. Bureau of Standards) described the study now being made by a special committee on the behavior of drain tile in alkali regions. This subject is treated on another page of this issue. A paper by G. P. Dieckmann (Northwestern States Portland Cement Co., Mason City, Iowa) gave the results of some tests on reinforced drain tile.

CONSTRUCTION

Three papers on construction were read, all by engineers representing some particular type of concrete forms or reinforcement. They were "Modern Concrete Roof without Forms," by J. E. Payne (General Fireproofing Co.); "Combination of Reinforcement and Forms in Concrete Construction," by T. W. Murray (Trussed Concrete Steel Co.); and "Steel Forms in Concrete Construction," by C. D. MacArthur (Blaw Steel Construction Co.). The first two described the use of ribbed wire mesh on either side of which concrete is plastered to form a wall or slab; the last paper took up in detail the steel forms made by the author's company.

STUCCO

The subject of stucco was very thoroughly considered at one session at which two papers and a committee report were presented. The first paper by J. C. Pearson (U. S. Bureau of Standards), was entitled, "Some Comparative Corrosion Tests of Plastered Metal Lath." It outlined a progress report on a series of tests started two years ago by the U. S. Bureau of Standards looking into the corrodibility of metal lath of different makes and composition, under different kinds of mortars and plasters, but all under the same condition of exposure, viz., as a test panel in a framework exposed to the normal outdoors weather of Washington, D. C. The panels have been in place two years and Mr. Pearson made a graded report on each of the hundred odd panels, but he states that while some slight changes are to be noted, he considers the time too short as yet to give a definite statement of conditions on which a method of construction or a metal composition might be based.

Cloyd M. Chapman (Westinghouse Church Kerr & Co.) read the other paper entitled, "Data on Lime Putty and Cream of Lime." In order to permit of a known amount of lime in field mixed cement-lime stucco mortar, Mr. Chapman made a number of tests on lime procurable at New York City, from which tests he was able to draw a curve between the number of gallons of water added to a barrel of lime and the pounds of lime per cubic foot of the cream of lime made. The curve and an explanation will appear in an early issue of ENGINEERING NEWS.

The Committee on Treatment of Concrete Surfaces presented a revised specification on "Stucco on Metal Lath," which included some minor changes in the one adopted last year, and a new specification for "Stucco on Wooden Lath." Both specifications were adopted.

CONCRETE ROADS

One morning of the week was devoted to the subject of concrete roads, and was essentially an overflow meeting from the concrete-road conference of the week before. Papers were read by H. J. Kuelling (County Highway Commissioner, Milwaukee, Wis.), R. C. Stubbs (Dallas, Texas), B. S. Pease (American Steel & Wire Co., Chicago, Ill.), and R. J. Wig (U. S. Bureau of Standards). Mr. Wig's paper was the same as the one read at the road conference (p. 446 of this issue) and brought out the most lively discussion of the session.

MISCELLANEOUS

The report of the Committee on Reinforced-Concrete Highway Bridges and Culverts is given in full elsewhere in this issue. The Committee on Reinforced-Concrete and Building Laws reported that the column tests on full-sized columns cast during actual construction of a building (see ENGINEERING NEWS, Jan. 29, 1914, p. 276) were to be made during the coming month and the results should be available soon. The Committee on Nomenclature reported progress toward the compilation of a glossary of concrete terms and was instructed to report for current publication in the *Journal* of the Institute.

Illustrated lectures on "The Use of Concrete in Hydraulic Works," by President Humphrey and "Progress and Development of Concrete Work on the Delaware, Lackawanna & Western R.R." by A. B. Cohen, Concrete Engineer of the railroad, took up most of one evening's session, and papers by form manufacturers on concrete fence posts took up practically all of another session. Other papers read were: "Layout of a Concrete Products Plant," E. S. Hanson (Chicago, Ill.); "The Problems of the Contractor," L. C. Wason (Boston, Mass.); "Sanitary Surfaces for Concrete Floors," T. Hugh Boorman (New York).

BUSINESS MEETING

The membership of the Institute has reached 891, but its expenses are still far in excess of its receipts and a satisfactory solution of its financial difficulties has not been reached. It is expected that the *Proceedings* of the Kansas City Convention, of March, 1912, will soon be published, while the 1913 *Proceedings* are being published in the monthly *Journal* of the Institute, now in its third month. It is hoped to increase the size, scope and importance of this *Journal*.

For the ensuing year the following ticket was elected by a unanimous vote of the 25 members present at the time: President, Richard L. Humphrey; Vice-President,

L. C. Wason (Aberthaw Construction Co., Boston, Mass.), Vice-President A. N. Talbot holding over on a two-year term. The even-year directors elected were E. D. Boyer (Atlas Portland Cement Co., New York), Robert W. Lesley (*Concrete-Cement Age*, Philadelphia), John B. Leonard (Consulting Engineer, San Francisco). Mr. Humphrey, who has been president of the association since its origin, announced that he would accept the presidency for the coming year only on condition that this should be his last year in the position.

Notes From Engineering Schools

UNIVERSITY OF IOWA—As a part of the state extension work of this University, the College of Applied Science held a "Municipal Lighting Convention" on Feb. 4, for the benefit of city engineers and other officials and in response to an expressed desire for authentic information on this subject. About 60 central-station men, city officials, etc., attended. A paper, "Rate Making for Public Utilities," was read by Dean W. G. Raymond; Prof. A. H. Ford gave an address on "Municipal Lighting" with demonstrations of the various methods now employed. C. P. Chase, President of the Iowa Engineering Co., presented the paper, "Municipal Ownership and City Managers."

NEWS NOTES

A Railway Bridge Was Wrecked by a derailed snow plow on the forward end of a train on the Moncton & Buctouche Ry., near Moncton, N. B., on Feb. 20. A portion of the train fell into the Shediac River, killing four trainmen.

Four Railroads Were Blocked by a Wreck on the Chicago, Milwaukee & St. Paul Ry., at Grand Crossing, La Crosse, Wis., on Feb. 16. According to press dispatches, the wreck of mail train No. 57 occurred at a switch used by the C. M. & St. P. Ry.; the Chicago, Burlington & Quincy R.R.; the Chicago & Northwestern Ry., and the Green Bay & Western R.R. Traffic on all four railroads was stalled for several hours.

The Ohio Flood Prevention and Water Conservation Bill, known as the "Conservancy Act of Ohio," abstracted on p. 322 of our issue of Feb. 5, passed the Senate promptly and was signed by the Governor on Feb. 17. Among minor amendments to the bill as abstracted in our issue of Feb. 5 was one requiring that freeholder's petitions must be signed by 500 instead of 100 freeholders and another providing that where a proposed district lies in two or more counties the proceedings for the formation of the district shall be conducted before a judge from each county included in the district.

The Miami Conservancy District, for flood prevention at Dayton, Ohio, and from there to Piqua and beyond on the north and to Hamilton and the Ohio River on the south, is proposed under the recent Conservancy Bill of Ohio. Hearings on the petition for the formation of the proposed district are advertised for Mar. 21. The district, if formed, will include ten counties.

The Water-Works of Berlin, Germany, supplied a total of about 19,000,000,000 U. S. gal. of water in the year ending Mar. 31, 1913, or about 24 U. S. gal. per capita per day for the 2,085,000 population served, within and without the city. Over 92% of the total supply was metered, but through only 32,000 meters (Mar. 31, 1913). The maximum daily per capita consumption was 1½ times the average and the minimum ⅞ of the average.

Exceptionally Heavy Rains in Southern California on Feb. 17-19 brought a recurrence of the flood conditions of Jan. 24-27 ("Eng. News," p. 328) in this section. According to press dispatches the precipitation near Los Angeles, averaged 7 in. The flooded area lies between the Tehachapi Mountains and the Mexican boundry line and embraces six counties. The floods caused much trouble on the railways and did much local damage to roads, farms and buildings.

Water-Works Improvements at Little Rock, Ark., have been ordered in court proceedings in accordance with findings of a Master in Chancery. A notable feature of the case is that instead of being a lawyer, the Master was an engineer in the person of Edward Flad, consulting engineer of St. Louis, Mo. Mr. Flad's findings were that the Arkansas Water Co., owner of the works in question, be required to build a dam and a 300,000,000-gal. "selective" reservoir, to store water from the Arkansas River when its turbidity and chlorine are low; a 14,000,000-gal. per day pipe line from the reservoir to the existing pumping station, a distance of about five miles; a settling reservoir of 2,500,000 gal. capacity, near the present settling reservoir on the hill south of the pumping station, and to provide 4,000,000 gal. additional filter capacity.

Work on the reservoir and pipe line must be started within three months, but if water of satisfactory quality is furnished meanwhile, only half the reservoir capacity need be provided at once; only half of the filter capacity need be made available until 1919 and the other half in 1923; and the settling reservoir need not be ready until 1927. By water of satisfactory quality is meant that practically free from chlorine and containing not over 400 parts per 1,000,000 of chlorine. The high chlorine is due to a tributary of the Arkansas, which is sometimes at flood while the main river is at a low stage. The plans for the works outlined must be approved by Mr. Flad.

The findings, as stated, are deemed essential to the fulfillment of the franchise of the water company. A new and more costly source of supply desired by the city, was ruled out as being inconsistent with the rates fixed in the company's franchise. George M. Gadsby is superintendent of the Arkansas Water Co. The headquarters of the company are in Pittsburgh. George W. Gibbs is chief engineer and W. Donaldson is sanitary engineer of the company.

The Alaska Railway Bill, providing for the construction by the United States Government of a railway from the coast to the Bering and Matanuska coal fields in the territory of Alaska, was passed by the House of Representatives, Feb. 18, by a vote of 230 to 70. The Senate has already passed a similar bill which provides funds for the work from a $40,000,000 bond issue. This part of the Senate bill was amended by the House of Representatives, whose bill provides for an immediate appropriation of $1,000,000, the balance to be paid by regular appropriations out of the treasury as the construction work progresses. The committees of two houses of congress, at this writing, are attempting to adjust the method of financing the work, which the recent action of the House of Representatives leaves no doubt will soon be commenced.

Standard Interstate Commerce Commission Railway Valuation Maps—A 22-page booklet just published by the Interstate Commerce Commission, Washington, D. C., gives the specifications for the preparation of the maps and profiles which must be filed with the commission to support the valuation of the property of railways. This pamphlet is now ready for distribution and should be added to the library of every engineer. The specifications cover tracing cloth, ink, size of sheets, scale, symbols, lettering, arrangement of data, indexing and data required on maps and profiles. An appendix contains a copy of sections of the law giving the Commission the authority to make a valuation of the railways, which refer to the methods to be employed by the Commission. A sample profile and samples of a right-of-way and track map, a station map of lands, and a station map of tracks and structures are included in an appendix B.

A Record of Deaths Due to Street Traffic has been compiled by the National Highways Protective Association, for a number of cities. According to these statistics Los Angeles, Calif., has the highest rate of automobile fatalities in the world. The rate there is 153 per million of population. Atlantic City, N. J., is second with 87; Utica is third with 70, and New York City is fourth with 64. The number of persons killed per million of population in the principal cities of New York and New Jersey, by street traffic, are as follows:

City	Automobiles	Street cars	Wagons
Utica, N. Y.	70	29	0
New York, N. Y.	64	23	2
Buffalo, N. Y.	54	47	21
Albany, N. Y.	40	40	10
Atlantic City, N. J.	87	0	0
Trenton	52	21	21
Paterson	48	8	16
Newark	47	18	21
State of New York	50	20	22
State of New Jersey	44	11	10

Building An 85-Ft. Chimney in seven days' working time is a feat accomplished by the Wiederholdt Construction Co., of St. Louis, Mo., on Feb. 15. The chimney was built on the company's system of reinforced-tile concrete with a reinforced-concrete foundation, and was erected at the plant of the Eggers Milling Co., of Hermann, Mo. The company informs us that the chimney was actually built, including its foundation, in seven days' working time and was completed and put in service just 16 days after the receipt of the order. During its construction there occurred, moreover, the heaviest snowfall recorded in ten years in St. Louis, and the temperatures were at no time higher than 20° F. when work was started in the morning. The sand was heated before mixing and the tile was heated before placing. Fires were kept in the chimney constantly with all openings closed. The chimney has a total height of 85 ft. and is 3 ft. in inside diameter.

Study of a Broken Rail—The Interstate Commerce Commission has just published a full report on a derailment caused by a broken rail in which three men were killed. The accident occurred on the Southern Railway near Oyama, N. C., Mar. 31, 1913, on a single-track line laid with 80-lb. rail, the track being in good condition. This part of the line is on tangent between two curves in opposite direction. The rail was found broken in eight places. Examination by the engineer-physicist of the Bureau of Standards, J. E. Howard, indicated that two of the fractures were caused by a train running in the other direction, presumably by a freight train which had passed eastward two hours before the wrecked train which was moving westward. One of the fractures showed a transverse fissure 1 in. in diameter. Chemical and tensile tests of the rail metal showed high phosphorus (0.09% to 0.10%) and brittle metal with elastic limit close to the ultimate and almost no elongation or contraction of area. Even after annealing, the elongation in 10 in. was only 0.9%. The material was 0.70% carbon and 0.80% manganese. The rail was rolled by the Tennessee Coal & Iron Co. in 1904, and has been in the tracks since January, 1905. Horizontal sections of the head and of the base at different depths in from the surface, polished and etched, showed longitudinal streakings; by cutting thin plates at these depths and bending them transversely the breaks occurred at the streaks.

New York State Barge Canal—Of the $77,651,948 worth of contracts placed for the construction of the New York State Barge Canal which has been in the course of construction for the past nine years, work to the value of $64,037,390, or 82% of the total work, was complete on Jan. 1, 1914. The contracts placed number 135, of which 79 have been finished. Of the 56 contracts still open, more than 50% of the work has been performed on 38, leaving (on Jan. 1) but 18 contracts upon which more than 50% of the work must still be done.

Contracts for the improvement of the Cayuga and Seneca Canal (for which work $7,000,000 was appropriated in 1909) have been awarded to the amount of $3,768,343. Work on these contracts to the value of $2,170,730, or 58%, was complete on Jan. 1, 1914.

For constructing terminals for the Barge Canal system throughout the state, 28 contracts were let, amounting to $1,863,105. On these contracts work valued at $917,642, or 49%, had been done on Jan. 1, 1914.

Studies of a Typhoid Outbreak at Rockford, Ill., in 1913 cleared the public water-supply of suspicion but led to some recommendations for changes in both the water-works and sewerage systems. In the last half of 1913, but more particularly from July to October, inclusive, there were an unusually large number of typhoid cases in Rockford. These followed sufficiently close upon the water-borne typhoid outbreak of 1912 to cause alarm. An investigation was made by Dr. C. E. Crawford, Chief Inspector, State Board of Health; Paul Hansen, Chief Engineer, Illinois Water Survey; H. N. Parker, Dairy Bacteriologist, University of Illinois; and Dr. W. E. Parks, Health Commissioner of Rockford. Various sources of typhoid infection were found, some responsible for but a few cases and others for many cases. About 95 cases appear to have originated in a single employee of a bakery and to have spread from this man to and through three milk routes. Although the water-supply was cleared of responsibility for the 1913 outbreak the investigators recommend:

> All connections between the public supply mains and the factory fire-protection systems should be abandoned . . . .
>
> The installation and continued use of Ford hydrants should be discouraged because the drain opening, provided on these hydrants for frost protection, permits polluted water from near the ground surface to be siphoned into the delivery pipe.

The investigators also recommended a comprehensive engineering study of the sewerage system and needs of Rockford with a view to rapid extensions of the sewers in accordance with a plan which will meet future needs of a sewage collection and at the same time convey the sewage to a point or points where it may be treated "so that Rock River and neighboring streams may be preserved in a reasonably clean condition." We are indebted to Mr. Hansen for a copy of the Rockford report.

The Status of the Engineer—"Engineers themselves do not definitely distinguish between the engineer that is to be and the engineer that is. Any instrument man or draftsman may call himself an engineer without fear of contradiction. There was once a foreman on construction at the St. Louis water-works, and he was not a good foreman at that, who posed among his acquaintances as an important member of the engineering force. A carpenter working on forms for concrete at the Chain of Rocks made his friends believe that he was engineer-in-charge of that construction; needless to say, his stay with us was short. So long as we, ourselves, do not brand the impostor among us, how can we expect the layman to tell us apart? This raises the question: What are the qualifications of an engineer?"—E. E. Wall, in "Journal of the Association of Engineering Societies," January, 1914.

Specifications for an Engineer's Dinner—The accompanying "specifications," together with a blueprint plan showing the seating arrangement, punch-bowl, etc., bound in a blue-paper folder like real specifications, formed the menu and souvenir of a farewell dinner given to a member of the City Engineer's staff of Waterbury, Conn., by his engineering associates. The original measures 4½x7 in., with a printed cover and typewritten "specifications." These specifications read as follows:

SPECIFICATIONS

FOUNDATION—The foundation shall be of Oysters on Half Shell laid in a bed of mortar composed of one part tobasco to three parts ketchup, the whole covered with a thin wash of Soup, which shall be placed before the foundation has obtained its initial set, to the end that it may, with the latter, form a compact and homogeneous mass. Crackers may be used as "plums" and wherever, in the opinion of the engineer, it is necessary, reinforcement of ½-in. round Celery shall be used.

BODY—The main portion of the work shall consist of Chicken Croquettes mixed in the proportion of 1:2½:5, Peas, Potatoes and Jelly being used sparingly, which, at the direction of the engineer, shall be subject to rigorous tests and inspection.

COPING—In placing the Waldorf Salad, the coarse aggregate of which shall be not larger than ½ in. in longest dimension, great care shall be used that the proper depth be obtained so that the Cheese, Crackers, Ice Cream and Cake may be properly worked into the mass and the whole form a solid and monolithic structure.

FINISH—At the completion of each section, it shall immediately be given a coat of heavy dark Coffee, and as soon as this has dried, the whole shall be gone over with a light wash of Punch, the latter to be subject at all times to the inspection and control of the engineer.

The tool used to give a smooth and even finish to the work shall be composed of Havana filler and wrapper. For this expert labor shall be furnished, as the appearance of the whole structure may be damaged by the employment of workmen who are unfamiliar with the use of these tools.

Field Bolts vs. Field Rivets—It is generally held that riveted connections in steelwork are distinctly superior to bolted connections. In fact, bolts are permitted only in mill-building frames and the like, and practically never in bridges or steel-cage building frames. The following account by W. T. Curtis of the Wisconsin Bridge & Iron Co. shows that field bolted connections may give excellent service; it relates to the old Manistee River viaduct of the Pere Marquette R.R., recently strengthened as described by Mr. Curtis in "Journal of the Western Society of Engineers," Dec. 1913:

> "The old viaduct was field-bolted (with the exception of buck and lateral bracing between the girders), a fact which was not discovered until after the field work of reconstruction had been begun. The owners on becoming aware of this condition at first seriously considered replacing all old field bolts with new rivets. Careful investigation showed the old structure to be in perfect alignment, with none of the bolts loose in the slightest degree, although the structure had seen years of service and had been somewhat overloaded in its later years, so it was decided not to go to the extra expense of replacing the bolts with rivets.
>
> As a matter of fact many of the buck and lateral connections which had been riveted originally were found to be in bad shape on account of loose rivets, the vibration of passing trains having worn deeply into some of the rivets. While it was realized that the points which had been riveted in the old structure were points subject to more service than the points which had been bolted, the investigation satisfied all who participated in it that bolts are in reality much better than they are generally considered in ordinary practice. Other observations we have had opportunity to make under somewhat similar circumstances have added evidence to this conclusion."

PERSONALS

Mr. Francis H. Sherrerd has been appointed Engineer of Street Repairs of Newark, N. J.

Mr. J. P. Rogers has resigned as Superintendent of the Spokane, Portland & Seattle Ry., Vancouver, Wash.

Mr. H. A. Boomer, formerly General Superintendent of the Lake Erie & Western R.R., has been promoted to be General Manager.

Mr. Frank S. Wheeler, recently Supervisor of the Erie R.R., at Deposit, N. Y., has been promoted to be Division Engineer.

Mr. Edward D. Levy, Assistant General Manager of the St. Louis & San Francisco R.R., has been appointed General Manager, succeeding Mr. W. T. Tyler, resigned.

Mr. Frank J. Evans, recently Trainmaster of the Atchison, Topeka & Santa Fé Ry., at Clovis, N. M., has been promoted to be Superintendent of the Pecos division, with headquarters at Clovis.

Mr. T. B. Coppage, recently Superintendent of the St. Louis & San Francisco R.R., at Fort Scott, Kan., has been promoted to be Superintendent of Transportation, with headquarters at Springfield, Mo.

Mr. H. Osborne, formerly Assistant Superintendent of Motive power of the Canadian Pacific Ry., at Montreal, Que., has been appointed Assistant Mechanical Superintendent, with headquarters at Montreal.

Mr. Leonard Day, Assoc. Am. Inst. E. E., has resigned as Chief of the Bureau of Fire Alarm Telegraph, New York City Fire Department, effective Mar. 1, in order to resume his practice as a patent expert.

Mr. Alfred W. Booraem, a former Deputy Police Commissioner of New York City, has been appointed Deputy Commissioner of Water Supply, Gas and Electricity for the Borough of Brooklyn, New York City, at a salary of $5500 per annum.

Mr. O. P. Wilson has resigned his position with the purchasing department of the Westinghouse Electric & Manufacturing Co., Pittsburgh, Penn., to become Assistant General Manager of the Norma Co. of America, 20 Vesey St., New York City.

Mr. J. L. White, formerly General Fuel Inspector of the Wabash R.R., has been appointed Assistant to the President of the Chicago, Indianapolis & Louisville Ry., Mr. F. A. Delano, M. Am. Soc. C. E., who recently resigned as President of the Wabash R.R.

Mr. J. E. Crawford, M. Am. Soc. C. E., former Bridge Engineer of the Norfolk & Western Ry., recently acting Chief Engineer, has been promoted to be Chief Engineer, succeeding Mr. C. S. Churchill, M. Am. Soc. C. E., appointed Chairman of the company's valuation committee, as noted in our isue of last week.

Mr. John A. Galvin, recently Architect of the Louisville & Nashville R.R., Louisville, Ky., has been appointed Senior Architect of the Chattanooga district of the valuation division of the Interstate Commerce Commission, with headquarters at Chattanooga, Tenn. Mr. Galvin is succeeded by Mr. J. C. Haley, formerly Assistant Architect.

Mr. Harry B. Ivers has resigned as General Manager of the Lewiston, Augusta & Waterville Street Ry., Lewiston, Maine, to become the head of the Frank Ridlon Co., of Lewiston. On the evening of Feb. 16, Mr. Ivers was the guest of some forty of his former associates and employees at a complimentary dinner. Mr. Ivers is succeeded as General Manager by Mr. E. T. Munger.

Naval Constructor Daniel C. Nutting, Jr., U. S. N., recently Construction Officer at the Puget Sound Navy Yard, has been appointed Superintendent of the Mechanical division of the Isthmian Canal Commission, with headquarters at Balboa, Canal Zone, Panama. He was born in 1869 and graduated from the United States Naval Academy at Annapolis in 1893, and afterward took a two-years' post-graduate course in naval architecture at the University of Glasgow, Scotland.

Mr. Frederick F. Schaller, recently with the Hall Switch & Signal Co., New York City, has been appointed Signal Engineer of the safety appliance division of the Interstate Commerce Commission. Mr. Schaller graduated from the Massachusetts Institute of Technology in 1896. His first signal work was with the Boston & Albany R.R. Subsequently he was Assistant Signal Engineer of the Lake Shore & Michigan Southern Ry. Since 1904 he has been with the Hall Switch & Signal Co.

Mr. Maxime Reber, former President of the Board of Public Improvements of St. Louis, Mo., has been appointed Consulting Engineer of the St. Louis City Plan Commission at a salary of $500 per month. His work will be to study the future transportation needs of the city and to recommend plans for better transportation connections and service with the present system of street railways. It is intended from his report to draw up proposed ordinances, to be submitted to the municipal assembly for passage, which will outline changes and additions to the street railway system and bring about a readjustment of traffic conditions.

Mr. Carl R. Gray has resigned as President of the Great Northern Ry., effective Mar. 1, and it is reported he will become President of the St. Louis & San Francisco R.R., under the projected reorganization of that company. Mr. Gray began his railway experience with the St. Louis & San Francisco R.R. as a telegraph operator in 1882, and he was continuously employed by this railway until 1911, when he accepted the presidency of the Spokane, Portland & Seattle Ry., one of the Hill lines. He was made President of the Great Northern Ry. in 1912. Mr. Gray was Senior Vice-President of the St. Louis & San Francisco R.R., when he resigned in 1911.

Mr. James E. Howard has resigned as Engineer Physicist of the United States Bureau of Standards to become connected with the bureau of inspection of safety appliances of the Interstate Commerce Commission under H. W. Belnap, Chief Inspector. Mr. Howard was born at Palmer, Mass., in 1851. About 1870 he was employed in the engineering department of the city of Boston, Mass. In 1873 he spent a few months on location survey and construction work for the St. Paul & Pacific Ry. in Minnesota. After five years as a Mechanical Engineer at Chicopee, Mass., specializing in the construction of testing machinery, he was appointed, in 1880, Engineer of Tests at the Watertown, Mass., Arsenal. Since 1910 he has been Engineer Physicist of the U. S. Bureau of Standards.

Mr. James K. Conner, First Assistant Engineer of the Lake Erie & Western R.R., has been promoted to be Chief Engineer, succeeding Mr. William G. Atwood, M. Am. Soc. C. E., resigned, as noted in our issue of last week. Mr. Conner was born at Wabash, Ind., in 1871, and was educated at Rose Polytechnic Institute. His railway engineering experience began in 1895 as Assistant Engineer and Superintendent of Track of the Cleveland, Cincinnati, Chicago & St. Louis Ry. Later he was Assistant Engineer of the Baltimore & Ohio Southwestern R.R., and of the New York Central & Hudson River R.R. From 1901 to 1906 he was a Designing Engineer with the Lake Shore & Michigan Southern Ry., except for short periods, 1903 to 1905, with the engineer corps of the Chicago & Southern Ry., and the Lake Erie, Alliance & Wheeling R.R. Since 1906 he has been First Assistant Engineer of the Lake Erie & Western R.R.

Mr. L. F. Loree, M. Am. Soc. C. E., President of the Delaware & Hudson R.R., has been elected President of the Wheeling & Lake Erie R.R., now in the hands of a receiver. Mr. Loree was born in 1858 and graduated from Rutgers College in 1877. The same year he entered the service of the Pennsylvania R.R. as an assistant in the engineer corps. For two years, 1881 to 1883, he was leveler, transitman and topographer on the preliminary and location surveys of the Mexican National Ry. Afterward he returned to the Pennsylvania R.R. as Assistant Engineer on the Chicago division of the lines west of Pittsburgh. He was promoted to be Engineer of Maintenance-of-way of this division, and later of the Cleveland & Pittsburgh division. In 1889 he was made superintendent of the same division. Subsequently Mr. Loree was promoted to be General Manager and Vice-President. In 1901 he was elected President of the Baltimore & Ohio R.R., which office he resigned in 1904 to become President of the Rock Island Co., and Chairman of the executive committee of the Chicago, Rock Island & Pacific Ry. and of the St. Louis & San Francisco R.R. Since June, 1906, he has been chairman of the executive committee of the Kansas City Southern Ry., and since 1907, President of the Delaware & Hudson Co.

OBITUARY

Frank H. Conner, a general contractor and President of the General Contractors' Association of Birmingham, Ala., committeed suicide Feb. 17.

Rudolph Link, Manager of Keuffel & Esser Co.'s western

office, at Chicago, Ill., died Dec. 23, 1913, at the age of 55 years. He was an engineering graduate of the University of Illinois.

Edward L. Pecham, a well known railway contractor of Blackwell, Okla., died Feb. 15. He was 54 years old. He was contractor for several hundred miles of railway construction in Oklahoma, which are now parts of the St. Louis & San Francisco R.R.

Elhanan Zook, former Superintendent of Construction of the Panama R.R., died Jan. 26, at Philadelphia. He graduated from the Pennsylvania Polytechnic College in 1875 and went to Nebraska, where he joined the engineer corps of the Chicago, Burlington & Quincy R.R. Subsequently he was Roadmaster and Trainmaster of this railway at Denver, Colo., and other places in the West. He entered the service of the Isthmian Canal Commission, in March, 1905, as Assistant Engineer under W. E. Dauchey. In 1906 he was transferred to the Panama R.R. and became Engineer of Maintenance-of-way.

John Coates, M. Inst. C. E., founder and chairman of the engineering firm of John Coates, Ltd., London, England, died Feb. 18. He was born at Newcastle-on-Tyne in 1846. His early work was with the Hawksley Co., gas and water-works engineers, of London. Eventually he established an engineering business of his own, which he expanded to include branch offices in Canada and Australia. In connection with his Canadian office at Ottawa he resided 16 years in that city and was connected with various public utilities throughout the Dominion. He was President of the Ottawa Gas Co. and the Ottawa Board of Trade.

Webster Gaxlay, M. Am. Soc. C. E., Engineer and Vice-President of the National Concrete Construction Co., of Louisville, Ky., died Feb. 17. He was born Feb. 17, 1863, in Louisville, and was educated in the public schools of that city. His engineering training and experience was received as an employee of the Louisville Water Co. Subsequently he entered business on his own account. In 1908, upon the death of Charles Hermany, Chief Engineer of the Louisville Water Co., Mr. Gaxlay was offered the position of Chief Engineer, but he refused to give up the business in which he had become so largely interested, although he agreed to act in a consulting capacity during the construction of the filtration plant. He was a charter member and a member of the Board of Direction of the Engineers' and Architects' Club of Louisville. He is survived by a widow and three children.

ENGINEERING SOCIETIES

COMING MEETINGS

NATIONAL BRICK MANUFACTURERS' ASSOCIATION.
Mar. 2-7. Annual meeting at New Orleans, La. Secy., Theo. A. Randall, Indianapolis, Ind.

VERMONT SOCIETY OF ENGINEERS.
Mar. 12. Annual meeting at Burlington, Vt. Secy., Geo. A. Reed, Barre, Vt.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.
Mar. 17-20. Annual convention at Chicago, Ill. Secy., E. H. Fritch, 900 S. Michigan Ave., Chicago.

HIGHWAY ENGINEERS' ASSOCIATION OF MISSOURI.
Mar. 18-20. Annual meeting at St. Joseph. Secy., Ray L. Gargill, St. Joseph, Mo.

IOWA STREET AND INTERURBAN RAILWAY ASSOCIATION.
Apr. 22-25. Annual meeting at Waterloo, Iowa. Secy., H. E. Weeks, Tri-City Ry. & Light Cos., Davenport, Iowa.

ILLINOIS WATER-SUPPLY ASSOCIATION.
Mar. 9-11. Annual meeting at the University of Illinois. Secy., Edward Bartow, Urbana, Ill.

ARKANSAS ENGINEERING SOCIETY.
Mar. 16-17. Annual meeting at Pine Bluff, Ark. Secy., P. B. Hill, Little Rock, Ark.

American Electrochemical Society—The spring meeting will be held in New York City, Apr. 16-18. There will be two topics: "Power for Electrochemical Purposes" and "Hydro-Electrometallurgical Processes." The Secretary is Joseph W. Richards, Lehigh University, Bethlehem, Penn.

Engineers' Club of Philadelphia—The officers for 1914 are: President, S. M. Swaab; Vice-President, J. A. Vogleson; Secretary, Henry L. McMillan, 1517 Spruce St., Philadelphia, Penn.

San Francisco Association of Members of the Am. Soc. C. E.—At the ninth annual meeting held on Dec. 19, the following officers were elected: President, C. H. Snyder, Vice-President, H. L. Haehl. The Secretary is E. T. Thurston, Jr., 713 Mechanics Institute, San Francisco, Calif.

Engineers' Club of Cincinnati—At a joint meeting of the Club and the local section of the American Society of Mechanical Engineers on Feb. 12, J. I. Lyle, of New York City, spoke on "Air Conditioning." The importance of maintaining a constant percentage of humidity in the air of public buildings and the use of air-purifying and humidifying machinery in many industries was explained. The Club's address is P. O. Box 333, Cincinnati, Ohio.

National Association of Port Authorities—The second annual meeting of the Association was held in New Orleans, Dec. 9-11, 1913. Eleven papers were presented bearing upon construction, administration and finance. The 1914 convention will be held in Baltimore, Md., Sept. 8-10. The Secretary is William J. Barney, Room 1200, 29 Broadway, New York City.

Iron and Steel Institute—The annual meeting of the Institute will be held at the Institution of Civil Engineers, Great George St., London, May 7-8, 1914. At this meeting the Bessemer Gold Medal will be awarded to Edward Riley, F. C. S. The annual dinner will take place on May 7. The autumn meeting will be held at Paris, Sept. 18-23. The Secretary is George C. Lloyd, 28 Victoria St., London, England.

Joint Meeting of Boston Engineering Associations—A dinner will be held at the City Club, Boston, on Mar. 4, to promote the social and professional interests of all engineers of Boston and vicinity. The organizations participating are the Boston Society of Civil Engineers, and the local sections of the American Society of Mechanical Engineers and American Institute of Electrical Engineers. The guests at the dinner will be Guy E. Tripp, Chairman of the Board, of the Westinghouse Electric & Manufacturing Co., who will speak on "The National Government and Business"; Dr. A. E. Kennelly, Professor of Electrical Engineering, Harvard University, whose subject will be "Results for the Electrical Engineering Fraternity Accomplished by the International Electrical Congress"; and Dr. M. W. Franklin, of the Sprague Electric Works, of Bloomfield, N. J., who will speak on "Ozone Applied to Ventilation." N. J. Neall, 12 Pearl St., Boston, is the Chairman of the Committee of Arrangements.

American Society of Engineering Contractors—The first annual meeting of the Chicago branch was held at the Auditorium Hotel, Chicago, Feb. 18. The chair was taken by H. F. Hackedorn (Indianapolis), First Vice-President of the main association, and the rules and by-laws were explained by T. Hugh Boorman, Treasurer of the association. The Chicago branch organization elected officers as follows: Chairman, De Witt V. Moore; Vice-Chairman, Carl Weber; Treasurer, C. W. Boynton; Secretary, James S. Campbel.; Directors, John T. Wallbridge and E. S. Hanson.

American Institute of Mining Engineers—The annual meeting was held in New York City on Feb. 16-17. After the address of welcome by President Charles F. Rand, on the evening of Feb. 16, an interesting smoker was held. On Feb. 17, a number of papers were presented ("Eng. News," Feb. 19, p. 438). The following officers were elected: President, Benjamin B. Thayer; Vice-Presidents, H. C. Hoover and W. L. Saunders; Secretary, Bradley Stoughton, 29 W. 39th St., New York City.

First Canadian and International Good Roads Congress—This Congress is scheduled to be held in Montreal, Que., May 18-23, under the joint auspices of the Canadian Highway Association, the Canadian Automobile Federation, and with the coöperation of the American Highway Association. The object of the Congress is to gain a full knowledge of the best methods of construction, maintenance and the administration of public roads in the Dominion.

Efficiency Society—The first national efficiency exposition and conference is scheduled to be held in New York City, Apr. 4-11, at the Grand Central Palace. Papers will be presented bearing upon the exhibits which will be of large variety. The exposition director of the Efficiency Society is situated at 41 Park Row, New York City.

American Society of Mechanical Engineers—A tentative program has been arranged for the coming spring meeting to be held in St. Paul and Minneapolis, June 16-19. Three groups of professional sessions will be held. It is expected that one session will be upon the subject of grain handling and ore handling, which is an important topic locally and an interesting one generally. Another session will be devoted to the use of powdered fuel and its efficiency and utility compared with oil and gas, for different purposes in industrial work (e.g., in openhearth furnaces, in cement kilns, in boiler furnaces, and for industrial furnaces). Reports will be presented by the Committee for Standard Specifications for Boilers and Other Pressure Vessels, and for Their Care in Service, and by the Committee on Flanges. The latter report was presented for preliminary consideration at the recent annual meeting in New York and provoked a spirited discussion.

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